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Journal of the
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CHARACTERISTICS OF REACTOR FUEL PROCESS WASTES[a]

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SYNOPSIS

The various types of high-level radioactive liquid wastes currently produced as effluents from reactor fuel processing plants are reviewed. Wastes expected from processes under development for power reactor fuels are also discussed. The wastes are characterized according to their chemical compositions, physical properties, and fission product activity levels.

This paper indicates the types and characteristics of the highly radioactive liquid wastes resulting from chemical processing of current and anticipated irradiated reactor fuels. Wastes containing only very small amounts of fission products, including certain gaseous wastes, may be released to the environment. However, the large volumes of radiochemical wastes of immediate concern contain large amounts of fission products and cannot be released to the environment because of the biological hazards. Unlike conventional industrial wastes and sewage, which may be rendered biologically harmless and then released, radiochemical wastes cannot be made less radiotoxic by any known treatment, and the practice has been to store the concentrated fission product solutions in large tanks interred at the surface of the earth. However, the growth of a nuclear power economy will involve increasing volumes of radioactive wastes, and such storage will become prohibitively hazardous.

First-hand familiarity with these wastes has until recently been limited to persons actively engaged in atomic energy operations. However, it has become increasingly evident that the assistance of specialists from many other

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fields of science and technology will be required before suitable methods for ultimate disposal can be developed and evaluated. The information given here is intended to introduce the disposal problems to interested persons in related fields.

Origin of Wastes

Irradiated reactor fuels are chemically processed to reclaim the unburned nuclear fuel and recover the transmutation products, such as Pu^{239} or U^{233}, from mixtures of the fission products and inert components of the fuel. At the present time, these are separated by solvent extraction (Fig. 1). While the processing details vary with the fuel being processed and the plant performing the operation, the basic principles are the same. Solid fuels, which have been stored 90-120 days following removal from the reactor (to permit

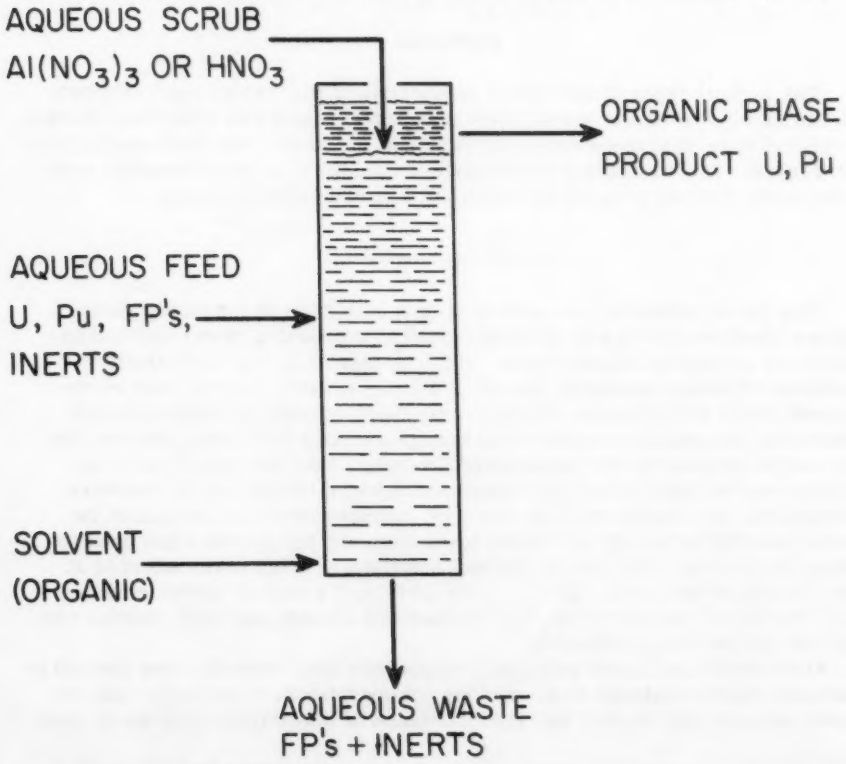

FIG. 1. SOLVENT EXTRACTION SEPARATION COLUMN

decay of short-lived fission product activities), are dissolved in nitric acid, and the solution is fed to the mid-section of an extraction column where it is contacted with an immiscible organic solvent that enters at the bottom. The solvent selectively extracts the uranium and plutonium; and an aqueous salt solution, added to the top of the column, scrubs the traces of fission products from the organic extract before it leaves the top of the column. The waste, containing more than 99.9% of the dissolved fission products, and inerts in the feed and scrub solutions, leaves at the bottom of the column. The uranium and plutonium are separated from each other and decontaminated further from fission products by additional treatment; but the waste from these steps contains very small amounts of activity and, in some cases, it recycled to the process.

Several processes in current use for processing irradiated fuels are shown in Table 1. It should be noted that the volumes given refer to the wastes as they come from the extraction column. In current practice, the wastes are evaporated to smaller volumes and are frequently neutralized before being stored in underground steel and concrete tanks.

Table I. Characteristics of Solvent Extraction Processes

Process	Application	Solvent	Salting Agent	Approximate Volume Untreated High-activity Waste
Purex	Pu-natural uranium	TBP in hydrocarbon	HNO_3	990 gal/metric ton U
Redox	Pu-natural uranium	Hexone	$Al(NO_3)_3$	1000 gal/metric ton U
Hexone-25	Enriched U-Al alloys	Hexone	$Al(NO_3)_3$	700 liters/kg enriched U
TBP-25	Enriched U-Al alloys	TBP in hydrocarbon	HNO_3 + $Al(NO_3)_3$	670 liters/kg enriched U
Thorex	Th-U^{233}	TBP in hydrocarbon	$Al(NO_3)_3$	1360 gal/metric ton Th

Characteristics of Present Wastes

Three types of irradiated fuels are currently being processed: (1) natural uranium, where plutonium and uranium are separated from fission products and from each other; (2) uranium-235–aluminum alloys where uranium-235 is recovered and separated from fission products; and (3) thorium, where uranium-233 and thorium are separated from fission products and each other. In each process the fuel is dissolved with nitric acid solutions, in preparation for the subsequent liquid-liquid extraction step. The solvents used for extraction are tributyl phosphate (TBP) or methyl isobutyl ketone (hexone). When TBP is used, it is diluted with a paraffinic hydrocarbon in order to obtain more favorable physical characteristics, and nitric acid is usually used as the salting agent in the aqueous scrub. This yields a waste (see below) that can be evaporated to much smaller volumes than is possible when aluminum nitrate is used as a salting agent, as in hexone extractions.

Table II gives the characteristics of the high-activity wastes produced by these processes. It should be emphasized that the numbers given are all approximations, the actual values depending on the operating characteristics of the particular chemical plant and the details of the chemical flowsheet being used. In addition to the concentrations of inert chemicals given here, all the

Table II. Characteristics of Current High-activity Wastes

Characteristic*	Purex	Redox	Hexone-25	TBP-25	Thorex
H, M	0.93	−0.3	−0.2	1.33	−0.05
Al, M	–	1.08	1.6	1.63	0.62
Na, M	–	0.23	–	–	–
NH_4, M	–	–	1.4	–	–
Hg, M	–	–	0.01	0.01	0.01
NO_3, M	0.93	3.05	6.0	6.2	1.8
F, M	–	–	–	–	0.039
Cr_2O_7, M	–	0.06	–	–	–
NH_2SO_3, M	–	–	–	0.04	–
Fe, Ni, Cr, g/liter	<1	<1	<1	<1	<1
Si, g/liter	–	–	<1	<1	–
PO_4, SO_4, g/liter	–	–	–	–	<1
Volume untreated	990 gal/ton U	1000 gal/ton U	700 liters/kg U	480 liters/kg U	1360 gal/ton Th
Specific gravity	1.03	1.16	1.25	1.25	1.10
Boiling point, °C	101	108	105	105	101
Freezing point, °C	−3	−18	−24	−24	−15
Specific heat	0.97	0.78	0.7	0.7	0.85
Vol after evaporation	60 gal/ton U	490 gal/ton U	510 liters/kg U	360 liters/kg U	380 gal/ton Th
Vol after neutralization	80 gal/ton U	830 gal/ton U	860 liters/kg U	610 liters/kg U	640 gal/ton Th

*Chemical composition is exclusive of fission products and heavy elements.

wastes contain fission products and lesser amounts of uranium, plutonium, and other heavy elements. The presence of these heavy elements is attributable mainly to process losses which usually approximate 0.1%.

Purex-type wastes[1] are mainly solutions of fission products in nitric acid and their physical properties are essentially those of about 1 M HNO_3. Trace constituents are iron, nickel, and chromium, present in varying amounts as products of corrosion of the stainless steel process equipment. Because of their low solids content, these wastes can be concentrated by evaporation to relatively small volumes and much of the nitric acid distilled and recovered for reuse in the process. Purex-type waste can be concentrated to about 8 M HNO_3, at which point oxidation of the fission product ruthenium to the volatile RuO_4 begins. The waste is then neutralized by addition of 50% NaOH solution and evaporated to near the solubility limit of the dissolved salts.

The wastes from the rest of the processes considered here contain significant amounts of aluminum. In the Redox process,[2] aluminum is added as a salting agent. In the processes for enriched uranium, it is present in the feed solution since it is a major component of the original fuel. In the Thorex process, it is used to provide what is known as an "acid deficient" condition, i.e., one in which the aluminum partially hydrolyzes to the monobasic or dibasic nitrates. The acid deficient solution is also used in the Redox and Hexone-25 processes.

The Redox waste contains some sodium, which is added in adjusting the feed solution to an acid deficient condition, and some dichromate, which is used to oxidize the plutonium to an extractable valence state. Ammonia is used to adjust for acid deficiency in the Hexone-25 process.

The Hexone-25 and TBP-25 wastes[3] have mercury, and the Thorex waste has both mercury and fluoride, which are added as catalysts in dissolving the fuel.

Ferrous sulfamate is added in the TBP-25 process scrub solution to reduce plutonium to a nonextractable valence state, thus effecting its separation with the fission products from uranium.

The trace constituents in these wastes include iron, nickel, and chromium as products of stainless steel corrosion, and, in some cases, small amounts of silicon and other impurities originally present in the fabricated fuel. Phosphate and sulfate are used in the Thorex process[4] to increase the decontamination of specific fission products and remove interferring ions.

The aluminum-bearing wastes are normally evaporated until the aluminum concentration reaches 2.2 M. If neutralization is desired, the pH is adjusted to about 13 by addition of 50% NaOH solution, 25-30% in excess of the stoichiometric amount required to form sodium aluminate.

Wastes from Power Reactor Fuel Processing

As mentioned earlier, it is through the growth of a nuclear power economy that we can expect a large increase in production of radioactive wastes over the remainder of this century. It is impossible to predict with assurance either the rate and magnitude of growth of such an industry or the characteristics of the wastes to be encountered. However, in order to permit higher reactor operating temperatures and more efficient recovery of heat, the fuels will probably be fabricated of high-melting metals and alloys, such as zirconium, molybdenum, and the stainless steels. Processes for recovery of such fuels are currently under development.

Although both aqueous and pyrochemical processing methods are being studied, it appears that in the immediate future processes will consist of methods for getting the fuels into aqueous solutions which can be processed by solvent extraction in the conventional matter. The application of large-scale non-aqueous or pyrochemical processing methods seem to lie in the more distant future.

As an indication of the aqueous wastes expected, based on current development work, processes for recovery of three types of power reactor fuels are

Table III. Power Reactor Fuel Recovery Processes

Process	Fuel	Process Description
Aqueous HF–TBP-25	High Zr–enriched U	HF dissolution, F complexing with Al, oxidation of U with Cr_2O_7, extraction of U with TBP
Sulfuric Acid–TBP-25	Stainless steel–enriched U	H_2SO_4 dissolution of SS, HNO_3 dissolution of U, extraction of U with TBP
Darex–Purex	Stainless steel–natural U	Dilute aqua regia dissolution, distillation of HCl, extraction of U and Pu with TBP

characterized in Table III. For recovery of enriched uranium from zirconium-uranium alloys of high zirconium content, it has been proposed to dissolve the alloy in HF, complex the fluoride by addition of aluminum nitrate, then extract with TBP.(3)

Stainless-steel-enriched uranium fuels can be processed by dissolving the steel with 4-6 M sulfuric acid and then dissolving the uranium in nitric acid.(3) The uranium would be extracted from the resulting solution with TBP. The Darex process,(5) a more recent development, is expected to be applicable to all stainless-steel-containing fuels. The uranium and stainless steel are dissolved in dilute aqua regia, after which the hydrochloric acid is distilled off nearly quantitatively, leaving a solution of stainless steel and uranium in nitric acid. The uranium and plutonium would be recovered by TBP extraction.

Table IV. Characteristics of High-Activity Power Reactor Fuel Processing Wastes

Characteristic*	Zirconium–25-TBP	Stainless Steel–25-TBP	Darex SS–28-TBP
H, M	1.0	3.4	2.0
Al, M	0.75	–	–
Zr, M	0.55	–	–
Fe, M	–	0.07	0.18
Cr, M	–	0.02	0.05
Ni, M	–	0.007	0.018
NO_3, M	2.3	2.7	2.3
F, M	3.2	–	–
SO_4, M	–	0.5	–
Cr_2O_7, M	0.01	–	–
Fe, Ni, Cr, g/liter	<1	–	–
Cl, g/liter	–	–	<1
Mn, P, Si, g/liter	–	<1	<1
Vol, untreated	1000 liters/kg U	495 liters/kg U	1760 gal/ton U
Specific gravity	1.2	1.1	1.2
Boiling point, °C	101	106	107
Freezing point, °C	Metastable <25°C	–6	–22
Specific heat	–	0.84	0.75
Viscosity, cp	2	1.2	1.3
Vol after evaporation	–	–	300 gal/ton (80 g SS/liter)

*Chemical composition is exclusive of fission products and heavy elements.

The high-activity wastes to be expected from processes such as these are given in Table IV. In addition to the chemical composition and volumes of untreated waste per unit of uranium processed, estimates are given of specific gravities, boiling and freezing points, specific heats, and viscosities. Because of their corrosivity and the chemical nature of the wastes, it is not certain whether evaporation will be feasible in many cases. Preliminary investigations show that the Darex-type wastes may be concentrated to stainless steel concentrations of 50 to 100 g/liter without resulting in instability. None of the wastes can be neutralized without precipitation of dissolved salts.

Fission Product Levels

Although the fission products are the source of most of the radiation hazard, they do not greatly influence the chemical properties of the wastes because they are present in such low concentrations. For every mega-watt-day of irradiation, about 1.1 g of fission products—both radioactive and stable—is formed. On this basis, it can be seen that most of the untreated wastes contain of the order of 1 g/liter of total fission products, consisting of isotopes of some 37 elements lying between zinc and dysprosium in the periodic table. While the total number of fission product atoms does not change after irradiation, the concentration of specific fission products does change with time, and for any given fuel is a function of the reactor power level during irradiation, the length of time the fuel has been irradiated, and the time elapsed since discharge from the reactor. The estimate of fission product levels and the nuclear properties of fission product mixtures is beyond the scope of this paper, but an extensive correlation of such information for thermal fission of U^{235} has recently been published.(6)

The contribution to the total beta activity by fission products of ten different mass numbers is presented in Figure 2. For the purposes of this illustration, natural uranium irradiated in a thermal neutron flux of 3×10^{13} $n/cm^2/sec$ for one year was chosen as a basis. At a decay time of 10 days, 63 d Zr^{95} and its daughter, N 35 d Nb^{95}, account for 20 percent of the total beta activity. They are followed in order of decreasing significance by 12.8 d Ba^{140} and 40 h La^{140} and by 290 d Ce^{144} and its 17.5 m Pr daughter. In the range of 100 to 1000 days decay, 290 d Ce^{144} - 17.5 m Pr^{144} represent 40 to 60 percent of the total activity. After this time, Sr^{90} and Cs^{137} and their daughters begin to assume an increasingly dominant role. In all, the isotopes considered here represent about 63 percent of the total activity after 10 days decay, 87 percent after 100 days decay, and greater than 98 percent at 1000 days and thereafter.

One of the properties of fission product mixtures of most interest to waste disposal is their heat emission. In any disposal scheme, provision must be made to dissipate the heat evolved as a consequence of radioactive decay if elevated temperatures which can lead to chemical instability, corrosion, and entrainment during storage are to be avoided. A typical curve of the fission product power expected in wastes from processing a fuel of high burnup is given in Figure 3. In this figure the specific fission product power (in watts/gal of waste) is plotted against the time of decay. It is assumed that 800 gal of waste has been produced from processing 1 metric ton of natural uranium, irradiated to 10,000 Mwd/ton at a specific power of 33 Mw/ton. Besides the total power, the contributions are noted for the rare gas fission products (normally separated when the fuel is dissolved), the rare earths, which are the major heat producers, and the long-lived hazards, Cs^{137} and Sr^{90}. After one day's decay, the fission power is rather formidable, 170 watts/gal; but after a 100 days' decay, a reasonable time to alloy before processing this fuel, the power has dropped to about 20 watts/gal. The power emission continues to fall until a decay time of about 3300 days, or nine years, the rare earths have begun to drop to a level where strontium and cesium and their daughters constitute the main source of power evolved in the waste. At this point the total power amounts to about 0.5 watt/gal and decreases according to the 27 to 28-year half-life of Sr^{90} and Cs^{137}.

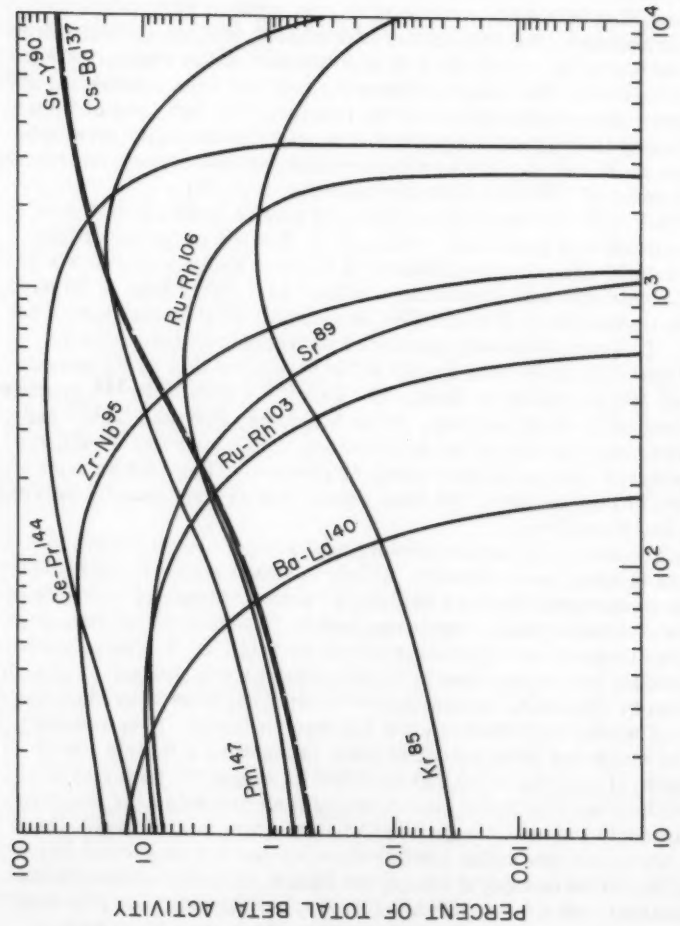

FIG. 2. DISTRIBUTION OF FISSION PRODUCT ACTIVITY AS A FUNCTION OF DECAY TIME. BASIS: IRRADIATION AT THERMAL FLUX OF 3×10^{13} $n/cm^2/sec$ FOR 1 YR.

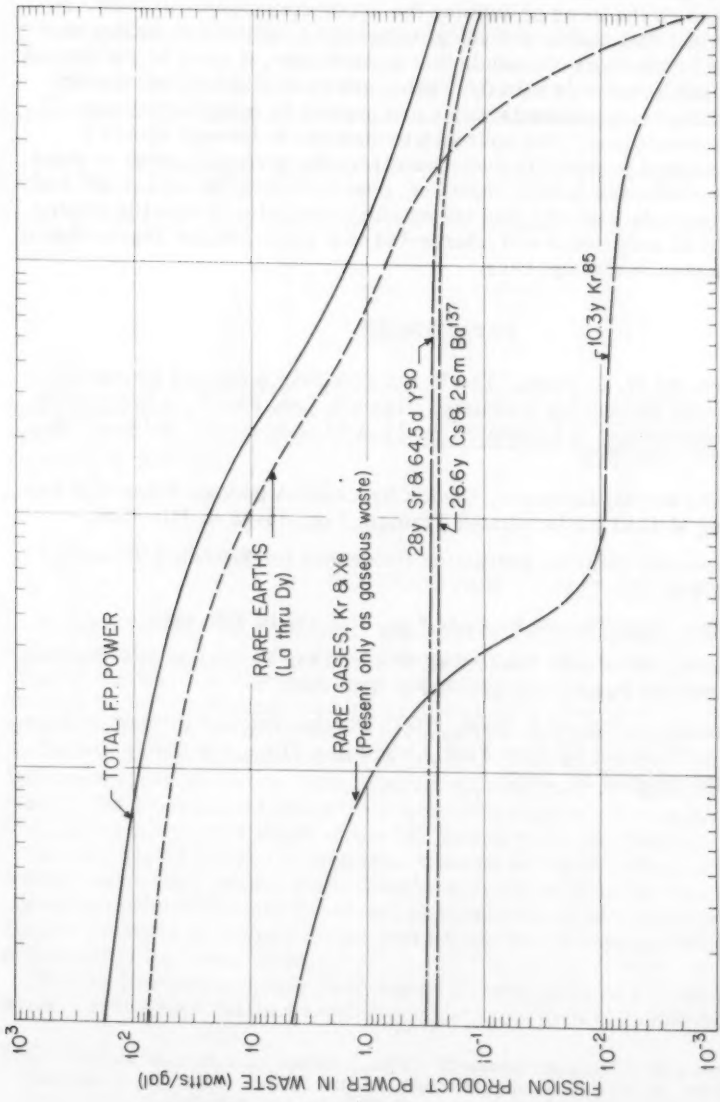
FISSION PRODUCT POWER IN WASTE (watts/gal)
TOTAL F.P. POWER
RARE EARTHS (La thru Dy)
RARE GASES, Kr & Xe (Present only as gaseous waste)
28y Sr 8 64.5d Y90
26.6y Cs 8 2.6m Ba137
10.3y Kr85
TIME OF DECAY (days)

FIG. 3. FISSION PRODUCT POWER IN WASTE AS A FUNCTION OF DECAY TIME. BASIS: 800 GAL WASTE PER METRIC TON U, IRRADIATED 10,000 Mwd/ton AT 33 Mw/ton SPECIFIC POWER.

CONCLUSION

This paper is concerned with the characterization of high-activity wastes from aqueous processing of irradiated reactor fuels. Characterization has been attempted on the basis of defining the major constituents present. Although the gross chemical and physical behavior of radioactive wastes may be controlled by the inert chemicals that predominate, it must be recognized that the hazards associated with the wastes are attributable to the fission products and the heavy elements which are present in relatively minute chemical concentrations. One approach to disposal is through chemical treatment designed to separate and concentrate the most hazardous of these isotopes. For effective hazard reduction, separations by factors of 10^6 and 10^7 would have to be attained, and the detailed chemistry of specific wastes would have to be understood and interpreted to a much greater degree than it has been possible to attempt here.

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RADIONUCLIDES IN REACTOR COOLING WATER[a]

Dade W. Moeller,[1] J.M. ASCE, George W. Leddicotte,[2] and Sam A. Reynolds[3]

ABSTRACT

This report summarizes studies made regarding the identification, source, and control of radionuclides in the recirculating type, cooling water system of the Low Intensity Test Reactor (LITR) at the Oak Ridge National Laboratory (ORNL).

INTRODUCTION

Reactor cooling water systems are basically of two types: single pass and recirculating. Typical of reactors employing single pass systems are those at the Hanford Works of the Atomic Energy Commission, the original Raleigh Research Reactor, and the proposed Test Tube Reactor.(1) Typical of reactors utilizing recirculating systems are the Low Intensity Test Reactor and the Oak Ridge Research Reactor at the Oak Ridge National Laboratory, the Materials Testing Reactor at the National Reactor Testing Station, the Pressurized Water Reactor at Shippingport, Pennsylvania, and the nuclear propulsion plants of the SSN Nautilus and SSN Skate.

Cooling water utilized in both type systems becomes radioactive. The resulting radioactive products are thought to originate from activation of dissolved solids and gases in the water, corrosion and scale formation, fuel and fission products escaping into the water from the reactor fuel elements, and activation of the water itself.

The purpose of this paper is to report a study made on a cooling water system of the recirculating type. Principal objectives of the study were to

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identify the radionuclides present in the water at various stages in the system, to determine the sources of these nuclides, and to measure the effectiveness of ion exchangers in their removal.

The Low Intensity Test Reactor

The investigations reported here were made on the cooling system of the Low Intensity Test Reactor (LITR) at the Oak Ridge National Laboratory. The LITR was built originally to serve as the mock-up for the Materials Testing Reactor. However, after being used for mechanical and hydraulic studies and later for criticality measurements, the mock-up was subsequently converted into a general experimental facility and has operated as such since March, 1951.

The LITR is a water moderated, beryllium reflected, heterogeneous, enriched uranium reactor. The fuel elements are of aluminum-uranium alloy with aluminum cladding. The reactor is cooled by a closed cycle recirculating system containing a total of approximately 10,000 gallons of water. The coolant enters the reactor tank at the top and is removed at the bottom, as shown in Figure 1. At the normal pumping rate of 1,200 gpm, the water is retained within the reactor core approximately 0.75 second.

A schematic flowsheet of the LITR cooling system is shown in Figure 2. Water leaving the reactor vessel is carried through an 8-inch lead-shielded pipe to a 7,000-gallon concrete shielded seal tank, the purpose of which is to allow decay of the very short-lived radionuclides, principally 7.4 s N^{16}, and to provide facilities for partial removal of dissolved gases. From the seal tank the water is pumped to air coolers and thereafter is returned to the reactor vessel for reuse.

Auxiliary facilities in the cooling system include a 600-gallon spray tank, a heat exchanger, and an ion exchange purification system. The spray tank serves as an emergency supply of cooling water to prevent melting of the core in case of accidental loss of water in the primary cooling system. The heat exchanger prevents freezing of the water in the cooling system during periods of shutdown in winter months and serves as an auxiliary cooling unit during summer months. The ion exchange purification system maintains the quality of water in the over-all system, approximately 3% ($\cong$ 40 gpm) of the coolant flow bypassing through it at all times.

The bypass purification system consists of three parts: a cation exchange bed, an anion exchange bed, and a mixed resin ion exchange bed. The normal flow of water through the bypass system is as follows: the inlet water is passed through a cloth type clarifying filter, the cation column, the anion column, the mixed-bed column, a second clarifying filter and then is returned to the primary cooling system. There are two cation and two anion columns, one of each type being in use at all times. There is only one mixed-bed column. Specifications for these units are as follows:

a) two columns, 24 inches in diameter by 46 inches high, each containing 8 cubic feet of IR-120 cation resin;

b) one column, 24 inches in diameter by 37 inches high, containing 6 cubic feet of IRA-410 anion resin and one column, 18 inches in diameter by 39 inches high, containing 3.5 cubic feet of IRA-410 anion resin;

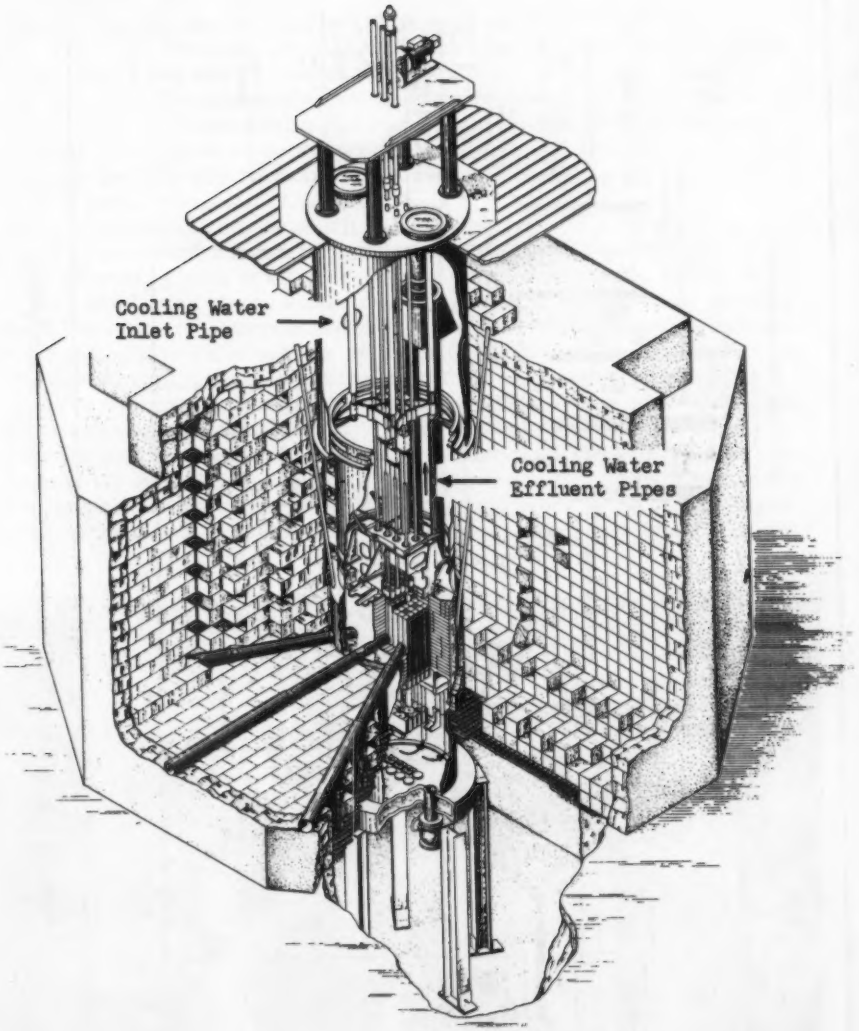

Fig. 1. Low Intensity Test Reactor: Core Vessel.

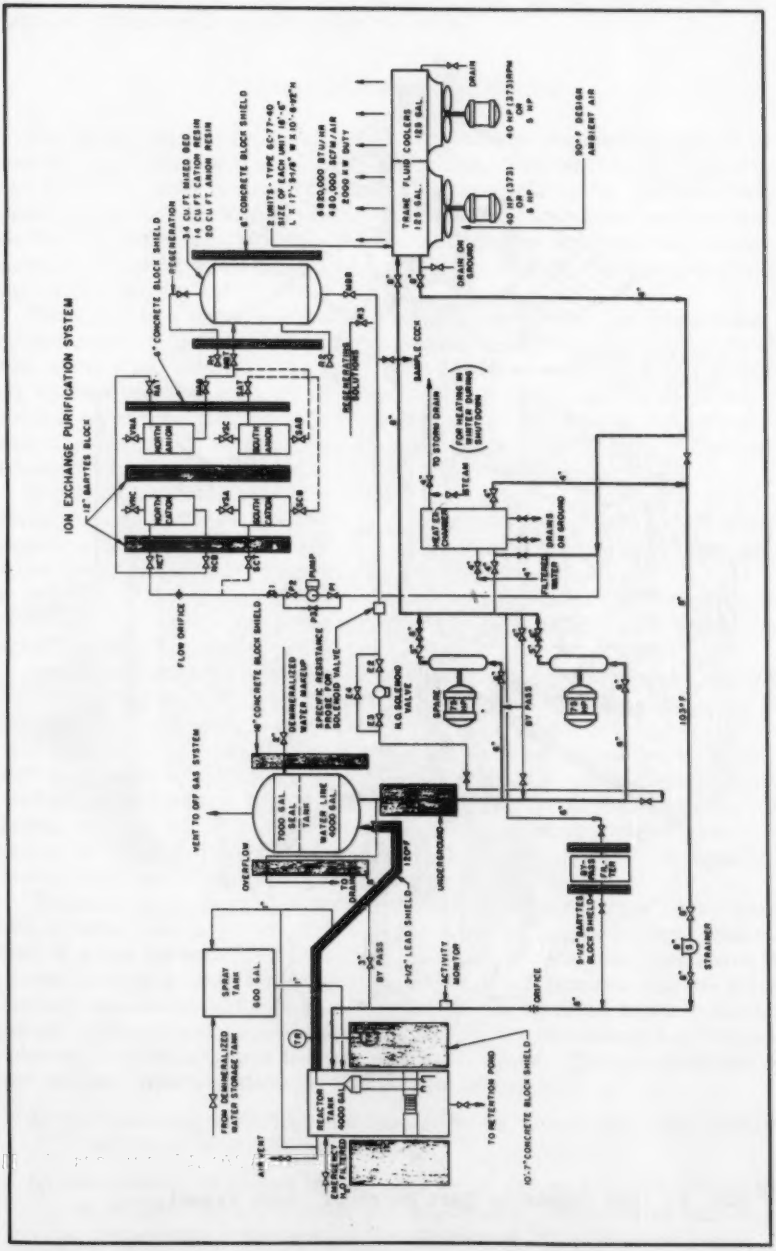

Fig. 2. Schematic Flowsheet: LITR Cooling Water System.

c) one mixed-bed column, 36 inches in diameter by 96 inches high, containing 20 cubic feet of IRA-410 anion resin and 14 cubic feet of IR-120 cation resin.

The cation units are shielded by rectangular walls of 12-inch barytes block. The anion units are shielded by 6-inch concrete block walls on three sides and share one wall of 12-inch barytes block with the cation units on the fourth side. The shielding serves also as weather protection. The units are covered by a conventional metal roof. A photograph of the cation and anion units, prior to construction of the shielding walls, is shown in Figure 3. The mixed-bed unit with its 6-inch concrete block shielding enclosure may be seen at the extreme left of the picture.

The gross gamma activity of effluent samples from each of the three resin columns is determined several times daily. Typical data on the range of removals effected by each of the columns are shown in Figure 4. It may be seen that, with freshly regenerated resin, the radioactivity in water passing through the cation column is reduced by a factor of approximately 5 and that passing through the anion column is reduced by a factor of approximately 15.

Regeneration of the cation and anion columns is based upon their effectiveness in removing gross gamma activity, a 50% decrease in effectiveness being considered an indication of column depletion. Regeneration of the mixed-bed unit is based on the specific resistance of its effluent, a decrease to or near the range of the resistivity of the water in the primary system being considered an indication of the necessity for regeneration. Under these operating conditions, it is necessary to regenerate the cation and anion beds

Fig. 3. LITR Cooling Water System: Ion Exchange Units

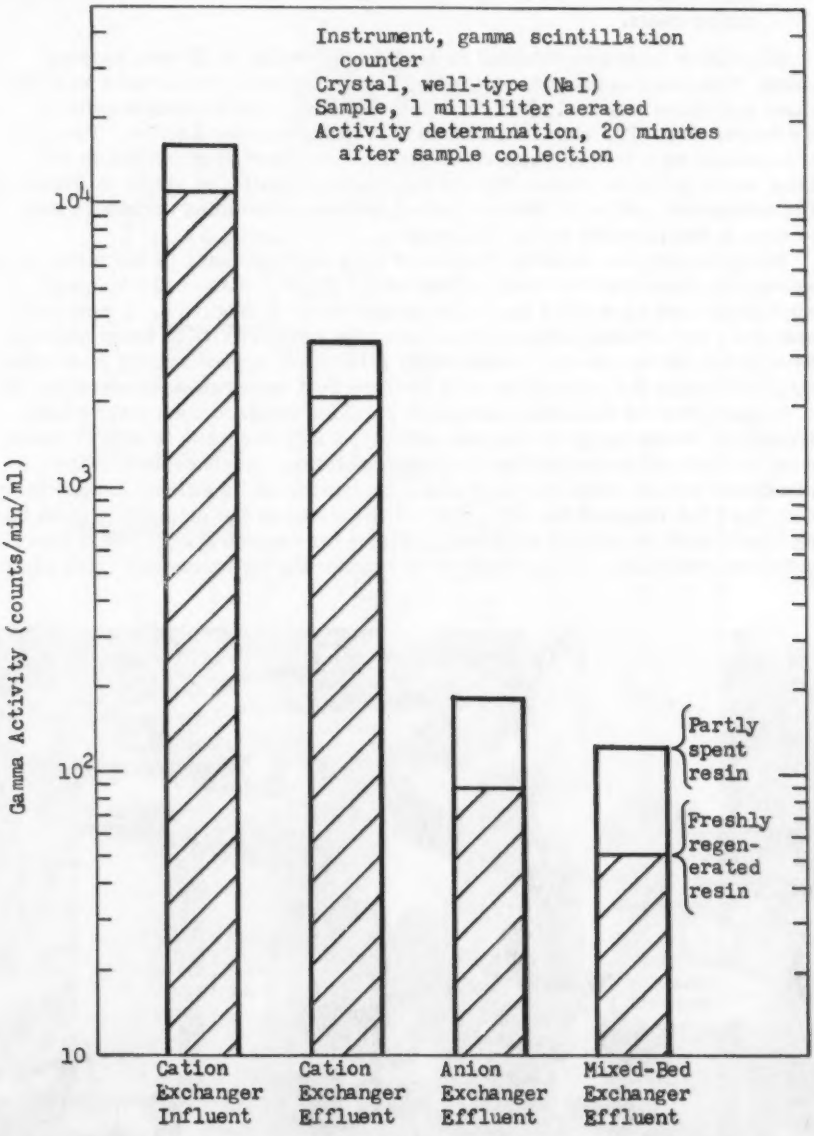

Fig. 4. Range of Gross Gamma Activity of LITR Cooling Water at Various Stages in Ion Exchanger Purification System.

approximately every four to six months. The average regeneration cycle for the mixed-bed column is about four weeks with an average of approximately 1,500,000 gallons of coolant passing through the unit per cycle. The average specific resistance of the primary coolant during this study was 810,000 ohm-centimeters.

Analytical Techniques

As previously stated, one of the principal objectives of this study was the identification of radionuclides in the cooling water at various stages in the system. The techniques employed are described below.

A. Radiochemical Assay

Because of the multitude of radionulcides present, it was necessary to chemically process the water samples to concentrate and separate the nuclides into groups whose identity could be established. The chemical procedure adopted represented a modification of that suggested by Lundell and Hoffman.[2] A flow sheet of the procedure is shown in Table I.

All samples to be processed were first aerated in a hood to remove dissolved radioactive gases. This procedure prevented unnecessary inhalation of radioactive gases by personnel processing the samples and also prevented entrapment and carry-down of these activities during subsequent separation procedures. After aeration, each sample

TABLE I

CHEMICAL SEPARATION SCHEME

1) ADD AN ALIQUOT OF THE RADIOACTIVE SAMPLE TO A 50 MILLILITER CENTRIFUGE TUBE. AERATE TO REMOVE RADIOACTIVE GASES. ACIDIFY WITH HNO_3 AND ADD Cl, Br, I, Cu, As, Fe, Y, Co, Mn, Zn, Ba, Sr, Ca, Na, AND K CARRIERS. PRECIPITATE Cl, Br, AND I WITH SLIGHT EXCESS OF Ag. HEAT TO COAGULATE THE PRECIPITATE. CENTRIFUGE.

2) PRECIPITATE: AgCl, AgBr, AgI, WASH WITH WATER, FILTER, MOUNT AND COUNT. DESIGNATE AS THE SILVER GROUP.

3) SUPERNATE: ADJUST ACIDITY TO 0.5 N HCl. SATURATE WITH H_2S. CENTRIFUGE.

4) PRECIPITATE: CuS + As_2S_3. WASH WITH WATER, FILTER, MOUNT AND COUNT. DESIGNATE AS THE ACID SULFIDE GROUP.

5) SUPERNATE: TRANSFER TO BEAKER. ADD 1 ml. CONC. HCl. BOIL UNTIL SOLUTION IS CLEAR. OXIDIZE Fe^{++} TO Fe^{+++} BY BOILING WITH HNO_3. COOL SOLUTION AND MAKE 1:1 WITH CONC. HCl. NEUTRALIZE WITH CONC. NH_4OH TO PRECIPITATE HYDROXIDE. CENTRIFUGE.

6) PRECIPITATE: Fe + Y HYDROXIDES. WASH WITH WATER, FILTER, MOUNT AND COUNT. DESIGNATE AS THE HYDROXIDE GROUP.

7) SUPERNATE: SATURATE WITH H_2S. CENTRIFUGE.

8) PRECIPITATE: CoS, MnS, ZnS. WASH WITH WATER, FILTER, MOUNT AND COUNT. DESIGNATE AS THE BASIC SULFIDE GROUP.

9) SUPERNATE: ACIDIFY SOLUTION WITH HCl. BOIL TO REMOVE H_2S. ADD $(NH_4)_2H(PO_4)$ AND AN EXCESS OF NH_4OH. DIGEST. CENTRIFUGE.

10) PRECIPITATE: Ca, Ba, AND Sr PHOSPHATES. WASH WITH WATER CONTAINING A FEW DROPS OF NH_4OH. FILTER, MOUNT AND COUNT. DESIGNATE AS THE ALKALINE EARTH GROUP.

11) SUPERNATE: Na + SOLUBLE IONS. TRANSFER ALIQUOT TO CULTURE TUBE FOR COUNTING. DESIGNATE AS THE SOLUBLE GROUP.

was acidified with nitric acid and isotopic and nonisotopic stable carriers were added. The resulting mixture was then processed to yield precipitates of silver chloride, bromide, and iodide (Silver Group), copper and arsenic sulfide (Acid Sulfide Group), iron and yttrium hydroxide (Hydroxide Group), cobalt, manganese, and zinc sulfide (Basic Sulfide Group), calcium, barium, and strontium phosphate (Alkaline Earth Group), and a soluble fraction (Soluble Group) containing those nuclides not carried by the stable carrier elements used.

Precipitation of the Hydroxide Group in the presence of a high concentration of ammonium chloride was found necessary to prevent manganese contamination of this group. It is possible, of course, to separate further each of the given chemical groups where necessary for positive identification on questionable cases. In several of the samples later discussed in this report, such techniques were employed.

Because of the time required for chemical processing of samples, it would be anticipated that the very short-lived radionuclides commonly reported in cooling water would not be detected in these studies.

B. Radioactivity Assay

After each chemical group separation, the sample was assayed by gross gamma decay and spectral studies. An interpretation of these data, together with knowledge of the chemistry of the particular group, became the basis for identification of the radionuclides in the sample.

Gross gamma decay studies were made using a well-type scintillation detector coupled to a research scaler. The detector was a NaI(T1) crystal two inches in length and two inches in diameter with a central well 1.5 inches deep by 0.86 inch in diameter. The well contained an aluminum liner which acted as an absorber for beta radiation emitted by the sample. All samples were mounted in 10 x 75 millimeter Pyrex culture tubes which fit down into the aluminum liner of the well-type crystal, permitting each sample to be essentially surrounded by the detector. A three-inch lead shield surrounded the crystal detector and sample.

Gamma spectral data were obtained by using spectrometers of both the single and 20-channel types. Detectors for these units were of the well-type previously described. Three radionuclides of known gamma energies were used for daily calibration of both spectrometers. There were Hg^{203}, Na^{22} and $Cs^{137} + Ba^{137}$.

Radionuclides in Litr Cooling Water

During routine operation, the LITR is shut down one day each week for fuel loading, changes in experimental setups, and other necessary alterations. To avoid nonequilibrium conditions, all samples were collected several days after the reactor had been brought back to full power.

A. Cation Exchanger Influent

The point chosen for initial sampling was the influent line to the cation exchanger. It was believed that such a sample would be representative of the coolant in general and would provide initial conditions for evaluation of the purification system. The time required for water

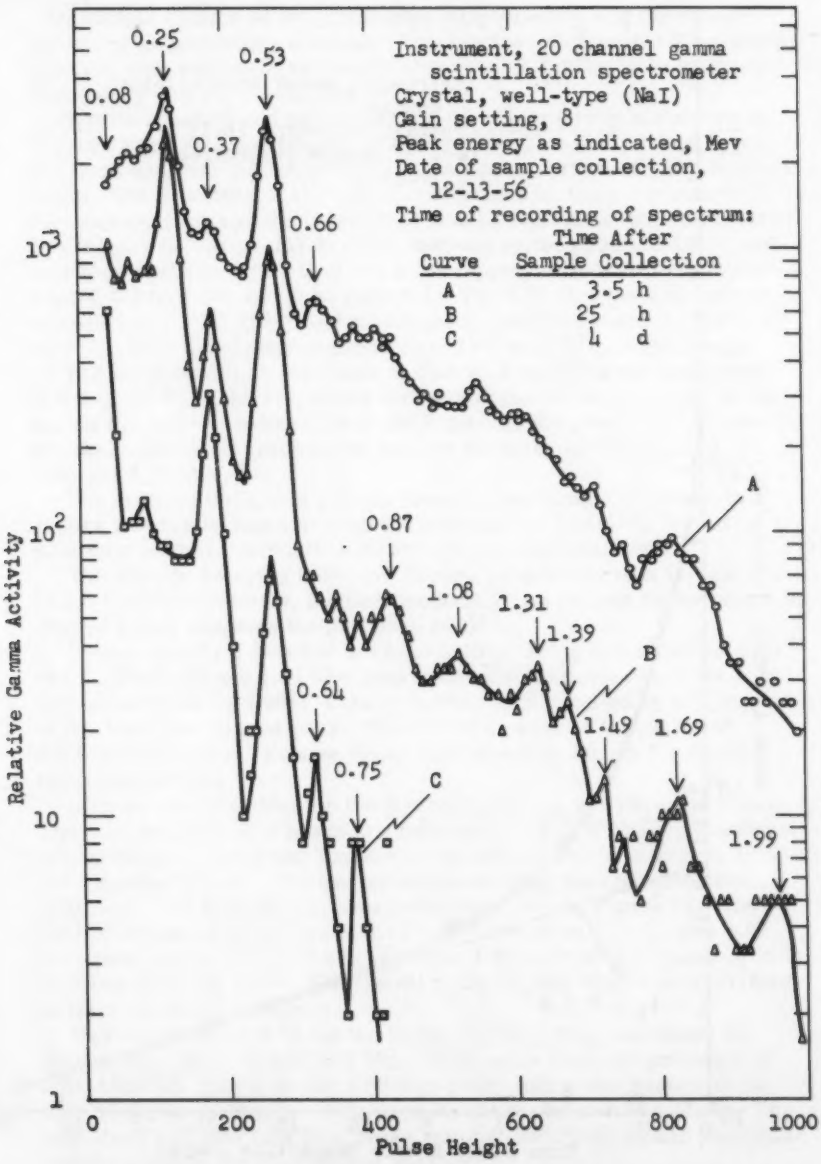

Fig. 5. Gamma Spectral Data for Influent to LITR Cation Exchanger: Silver Group.

SA 2

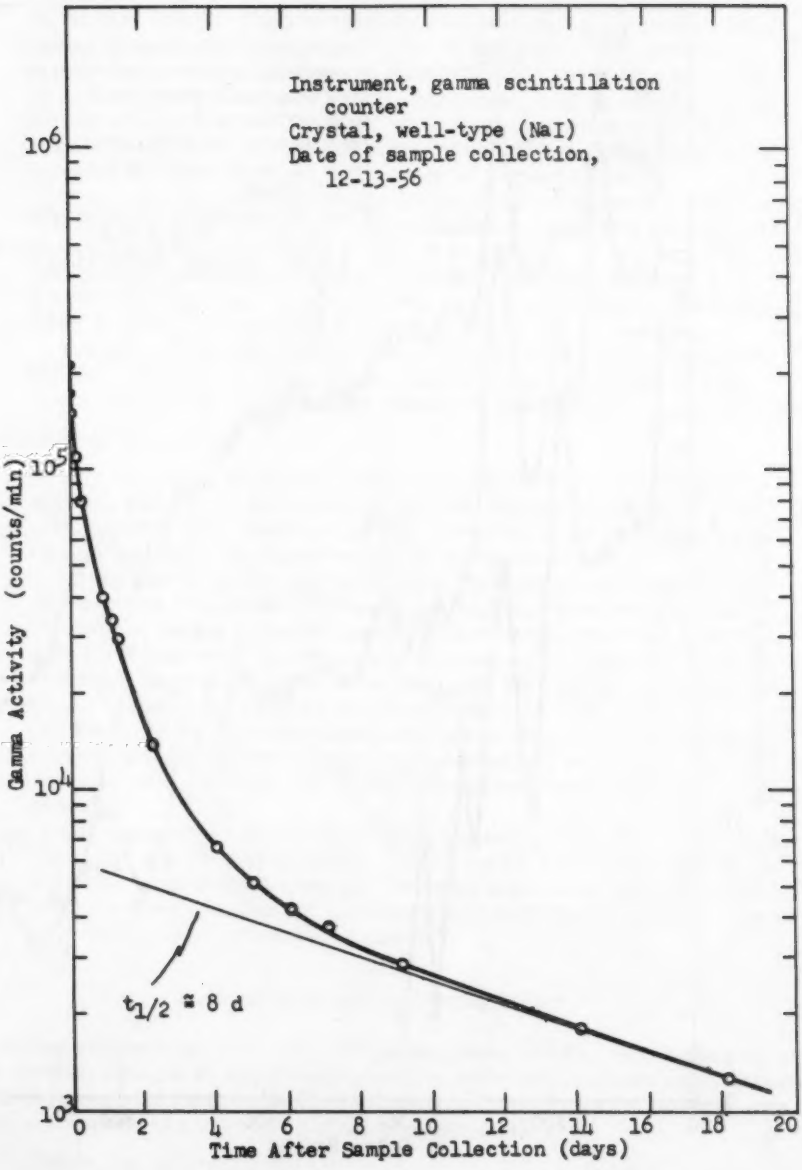

Fig. 6. Gross Gamma Decay for Influent to LITR Cation Exchanger: Silver Group.

leaving the reactor vessel to reach this initial sampling point was 5 to 10 minutes.

A sample volume of 140 milliliters was collected and chemically separated as previously outlined. For spectral studies, the 20-channel spectrometer was used, the single channel unit serving only as a supplementary unit for "spot" checks.

Gamma spectral and decay data for the Silver Group are shown in Figures 5 and 6. The spectral data show that 21 h I^{133}, 8.05 d I^{131}, and their daughter products constitute a major portion of this chemical group. The presence of I^{133}, is characterized by the prominent 0.53 Mev gamma peak and the lesser 0.87 and 1.31 peaks as shown in Curve B of Figure 5. Iodine-131 is characterized by the prominent 0.37 and 0.64 Mev peaks in Curve C of the same figure, as well as the approximately 8 d half life shown in Figure 6. The 0.06 Mev peak is representative of 5.27 d Xe^{133} and its daughter Cs^{133m} as well as 4.8 x 10^{-10} s Xe^{131m}1, daughter products of I^{133} and I^{131}, respectively.

The prominent 0.25 Mev peak in Curves A and B is characteristic of 9.2 h Xe^{135} + Cs^{135m}, which strongly suggests the presence of its parent 6.7 h I^{135}. Iodine-135 is confirmed by the presence in Curbe B of peaks near its characteristic gamma energies of 0.84, 1.13, 1.28, 1.46 and 1.72 Mev.(3)

The rapid initial gross gamma decay of the sample as shown in Figure 6 strongly suggests also the presence of 52mI^{134}. However, its presence would be difficult to confirm by the spectral data.

The rapidly decaying 0.66 Mev gamma peak in Curve A is indicative of 2.3 h I^{132}. Similarly, the intermediate gross gamma decay shown in Figure 6 also suggests the presence of I^{132}.

Gamma spectral data for the Acid Sulfide Group are shown in Figure 7. The 0.51 and 1.03 Mev peaks indicate the presence of the positron emitter 12.8 h Cu^{64}. This is further substantiated by the presence of the 1.34 Mev gamma peak. The 0.14 Mev peak is indicative of 6.0 h Tc^{99m}. Gross gamma decay data shown in Figure 8 confirm these indications.

Gamma spectral data for the Hydroxide Group are shown in Figure 9. The 0.10 and 0.22 Mev peaks are indicative of 2.33 d Np^{239}. Confirmation is given by the gross gamma decay data as shown in Figure 10 and the subsequent single channel spectrum showing the characteristic 0.09, 0.21, and 0.28 Mev gamma peaks as shown in Figure 11. The 0.53 Mev peak is indicative of 1.87 h F^{18} and 50 m Y^{91m}. The 0.83 Mev peak represents 2.58 h Mn^{56}. The 1.36 Mev peak is possibly indicative of 15.0 h Na^{24}. Yttrium-91m, Mn^{56}, and Na^{24} are confirmed in later chemical groups.

Gamma spectral data for the Basic Sulfide Group are shown in Figure 12. The 0.10 and 0.22 Mev peaks again show the presence of 2.33 d Np^{239}. The 0.84 and 1.78 Mev peaks and gross gamma decay data as shown in Figure 13 confirm the presence of 2.58 h Mn^{56}. The long lived 1.16 and 1.34 Mev peaks and the chemistry of this particular sample confirm the presence of 5.2 Co^{60}.

Gamma spectral data for the Alkaline Earth Group are shown in Figure 14. The 0.10 and 0.22 Mev peaks of Curve A are again indicative of 2.33 d Np^{239}. Half life studies of the 0.56, 0.76 and 1.05 Mev

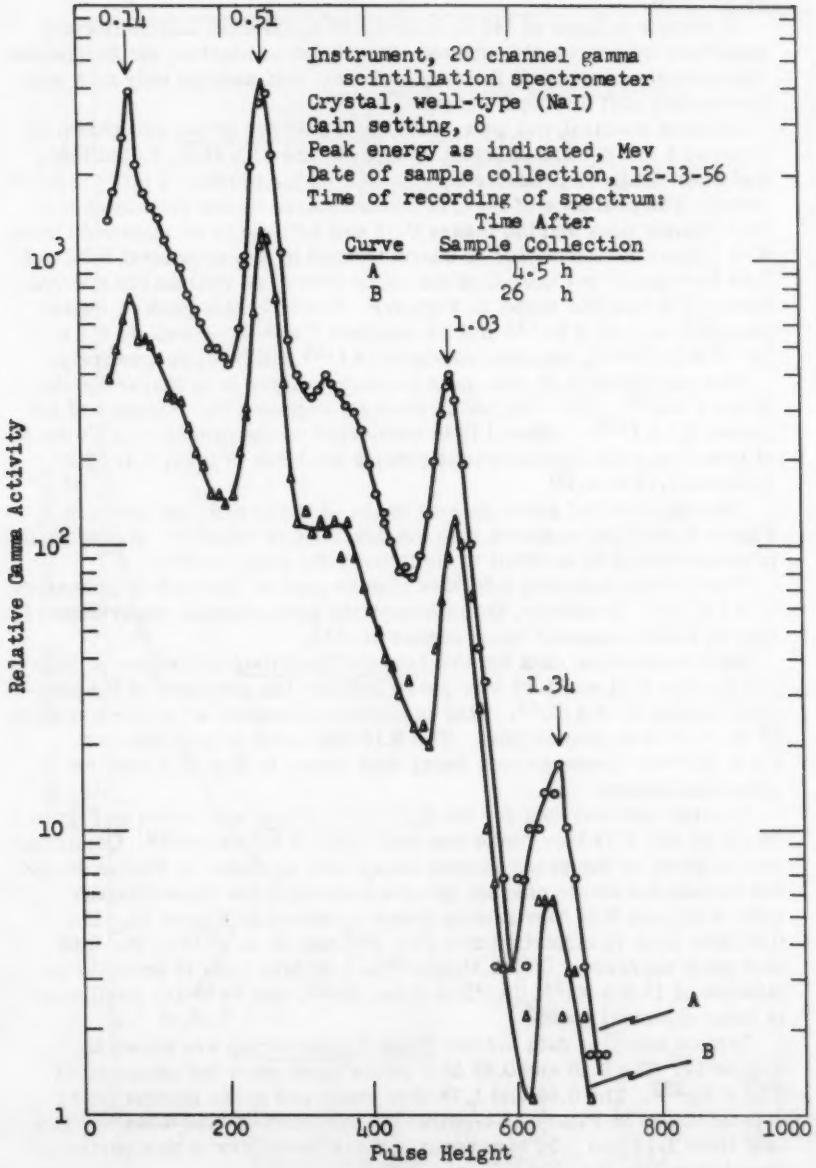

Fig. 7. Gamma Spectral Data for Influent to LITR Cation Exchanger: Acid Sulfide Group.

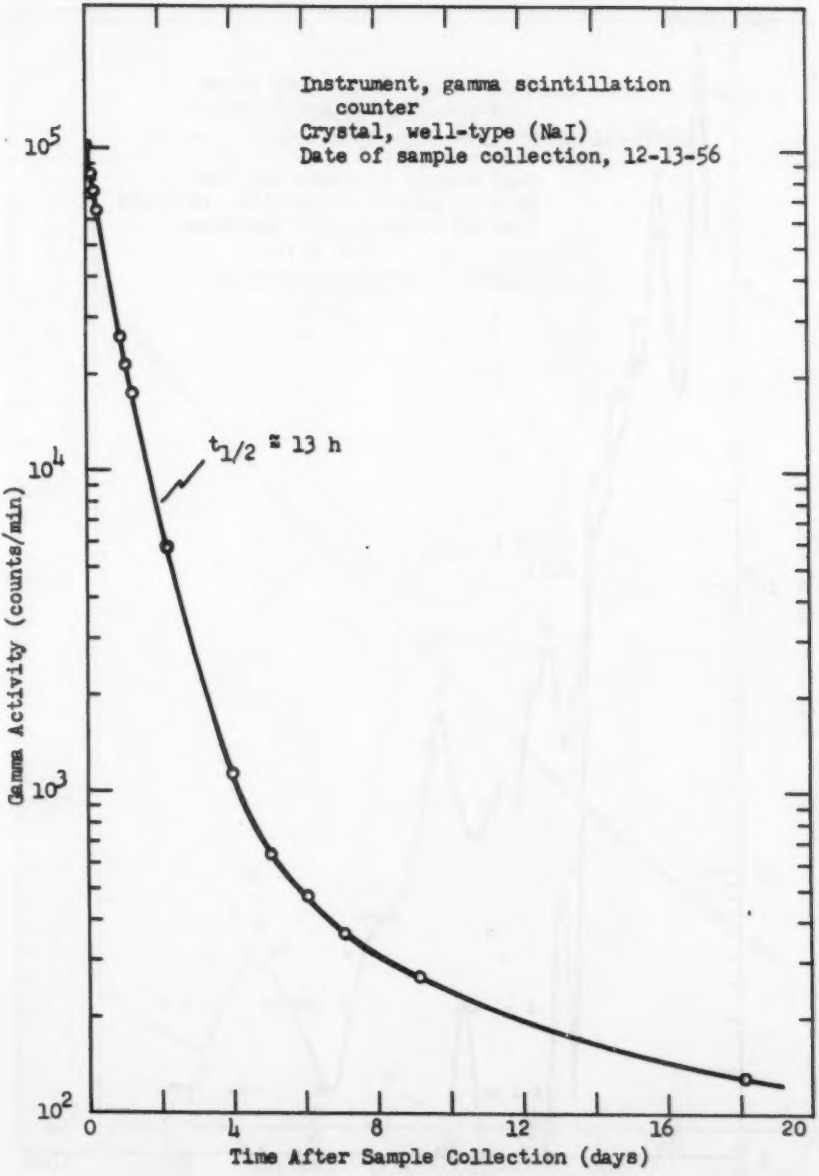

Fig. 8. Gross Gamma Decay for Influent to LITR Cation Exchanger: Acid Sulfide Group.

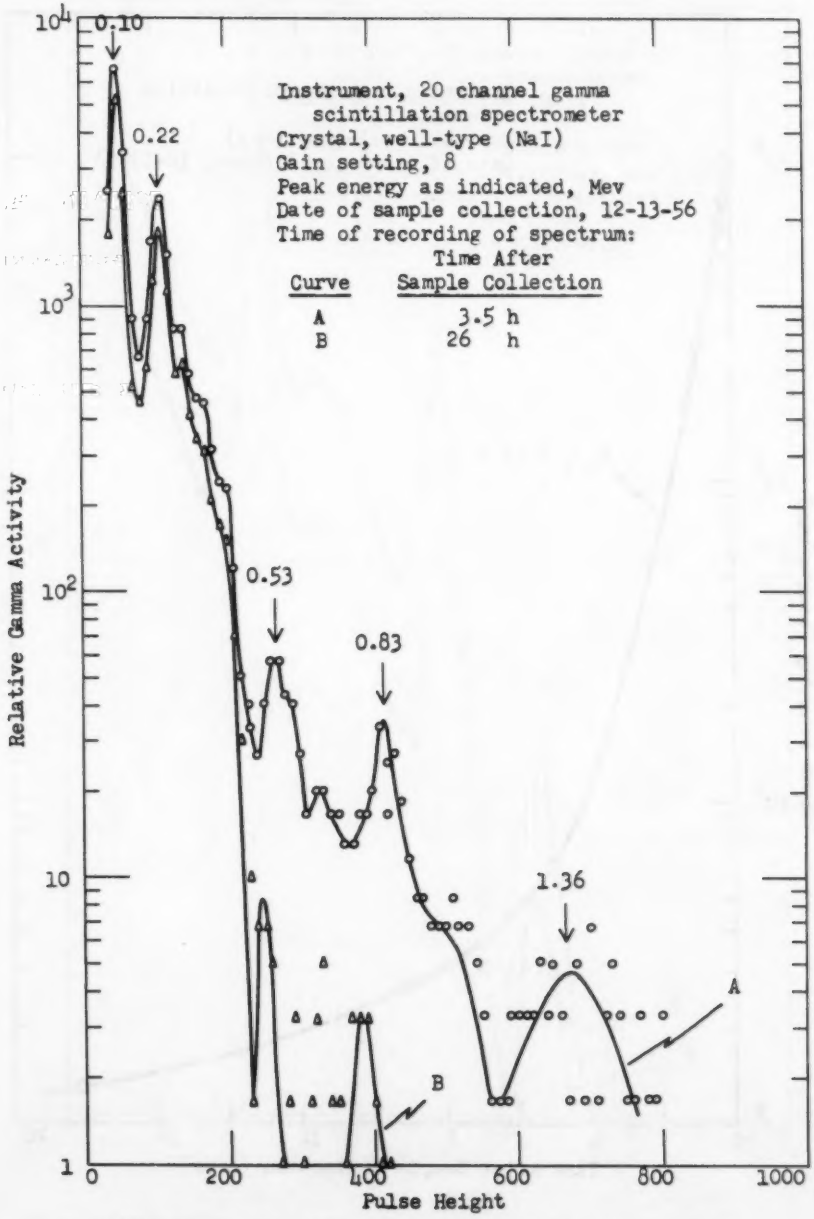

Fig. 9. Gamma Spectral Data for Influent to LITR Cation Exchanger: Hydroxide Group.

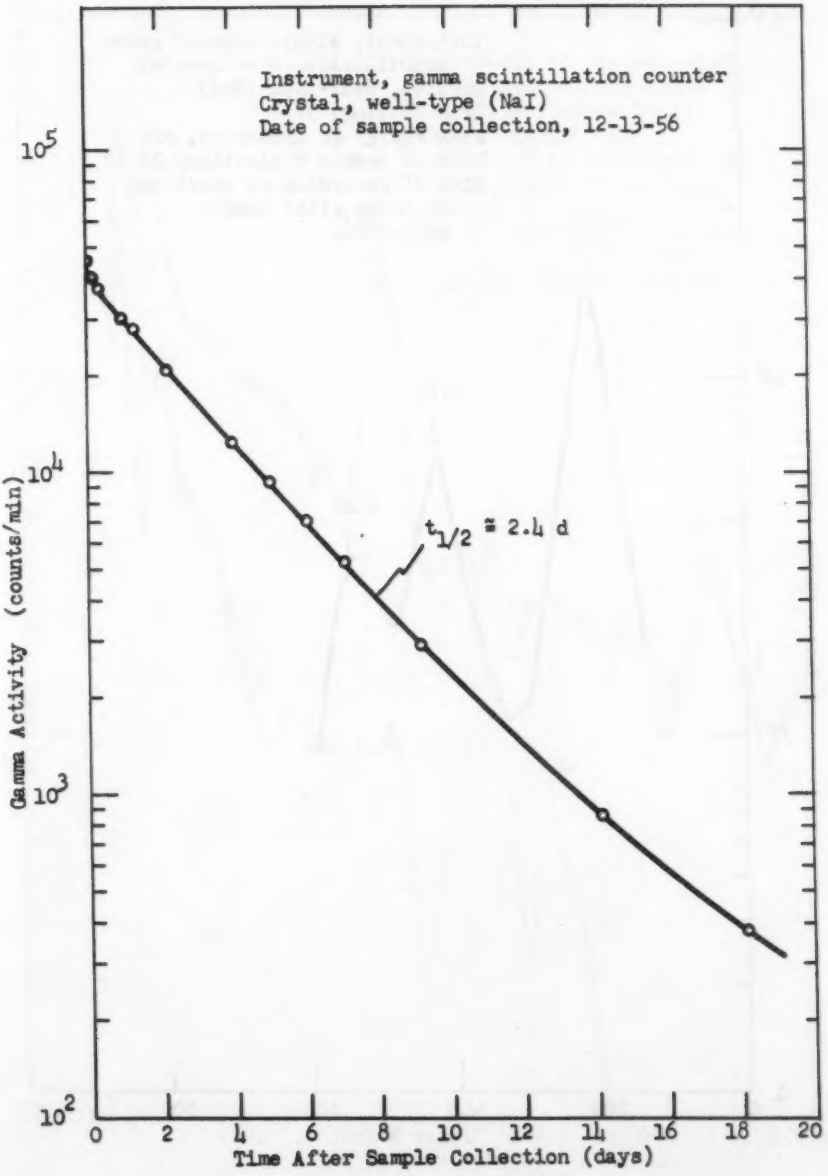

Fig. 10. Gross Gamma Decay for Influent to LITR Cation Exchanger: Hydroxide Group.

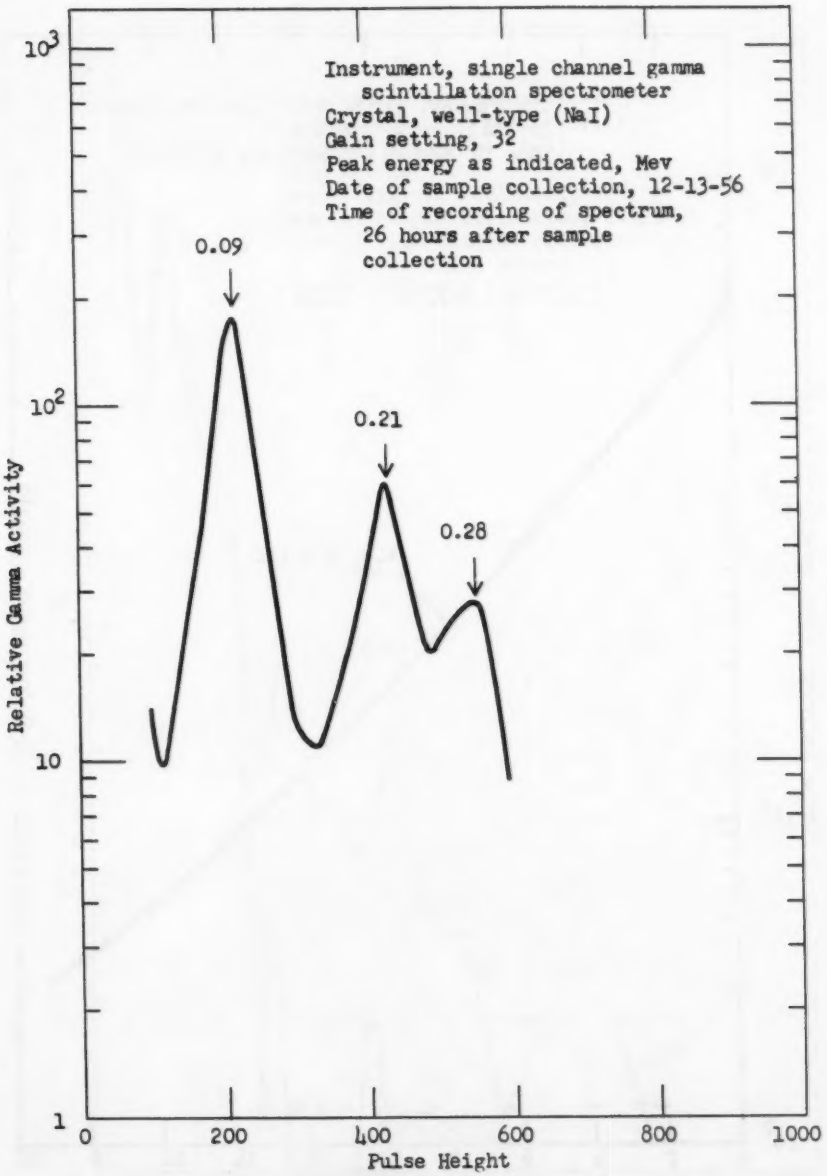

Fig. 11. Gamma Spectral Data for Influent to LITR Cation Exchanger: Hydroxide Group.

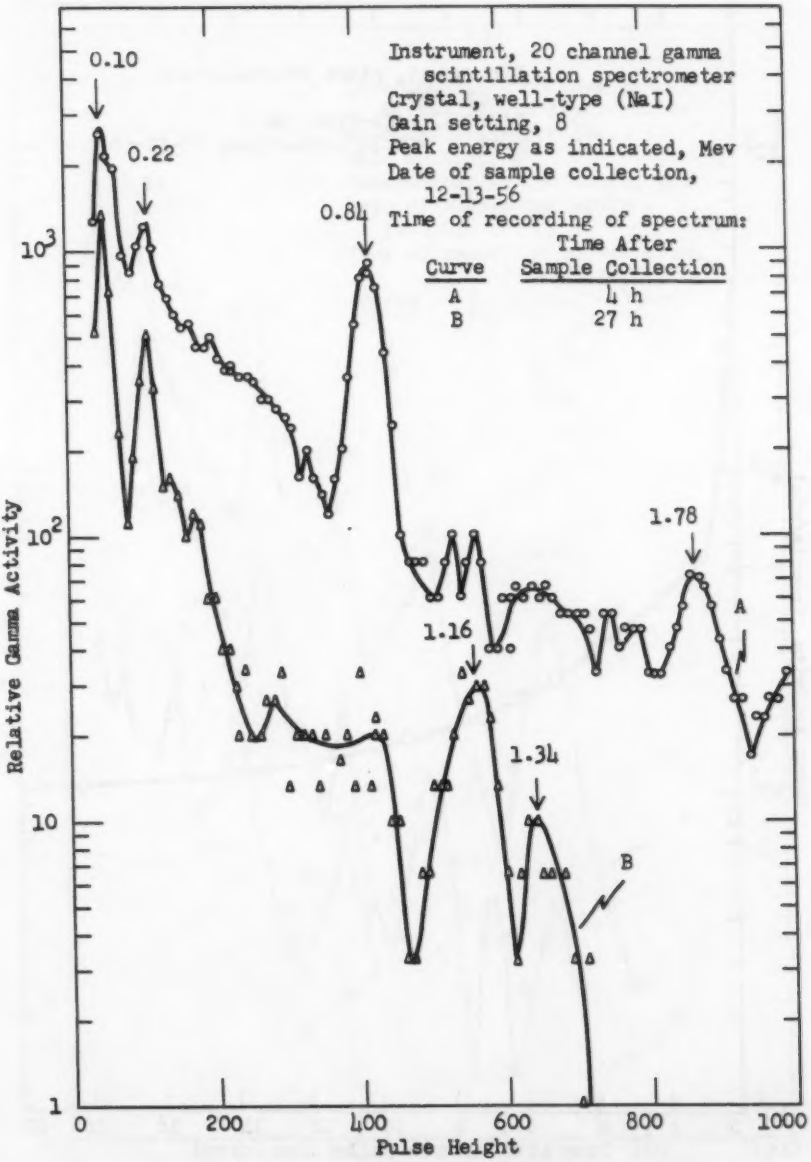

Fig. 12. Gamma Spectral Data for Influent to LITR Cation Exchanger: Basic Sulfide Group.

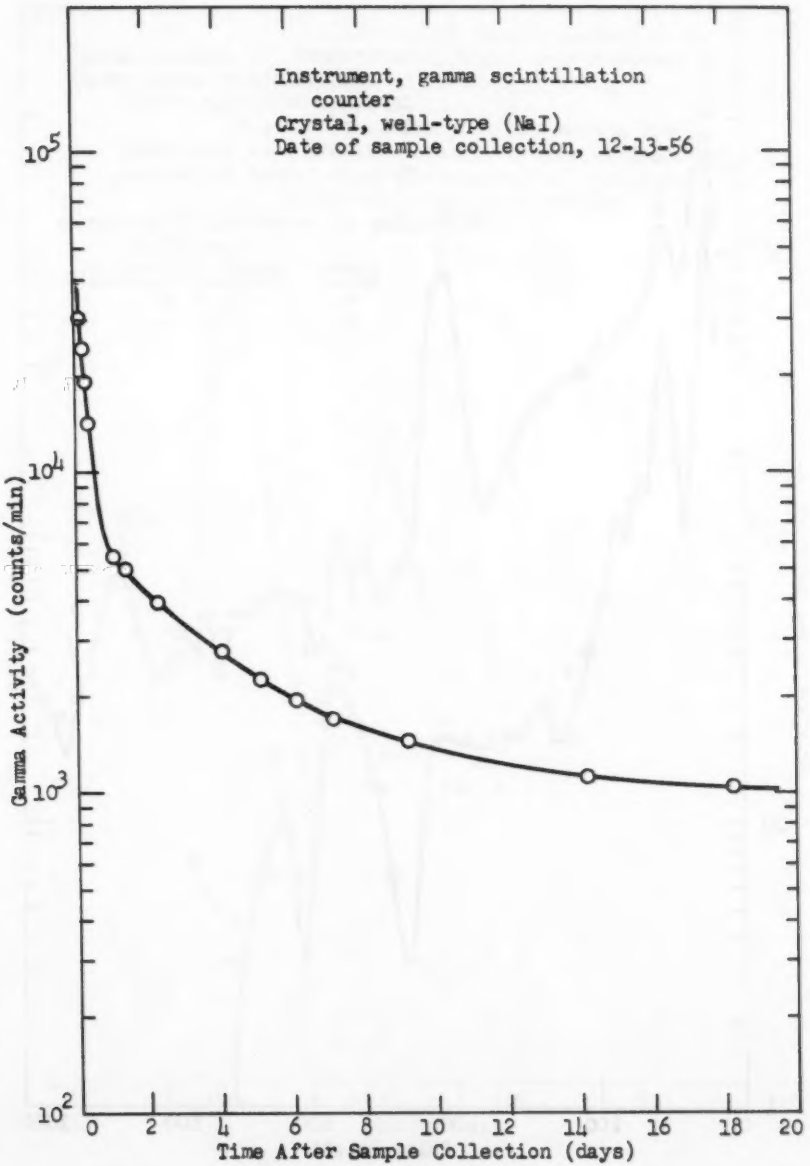

Fig. 13. Gross Gamma Decay for Influent to LITR Cation Exchanger: Basic Sulfide Group.

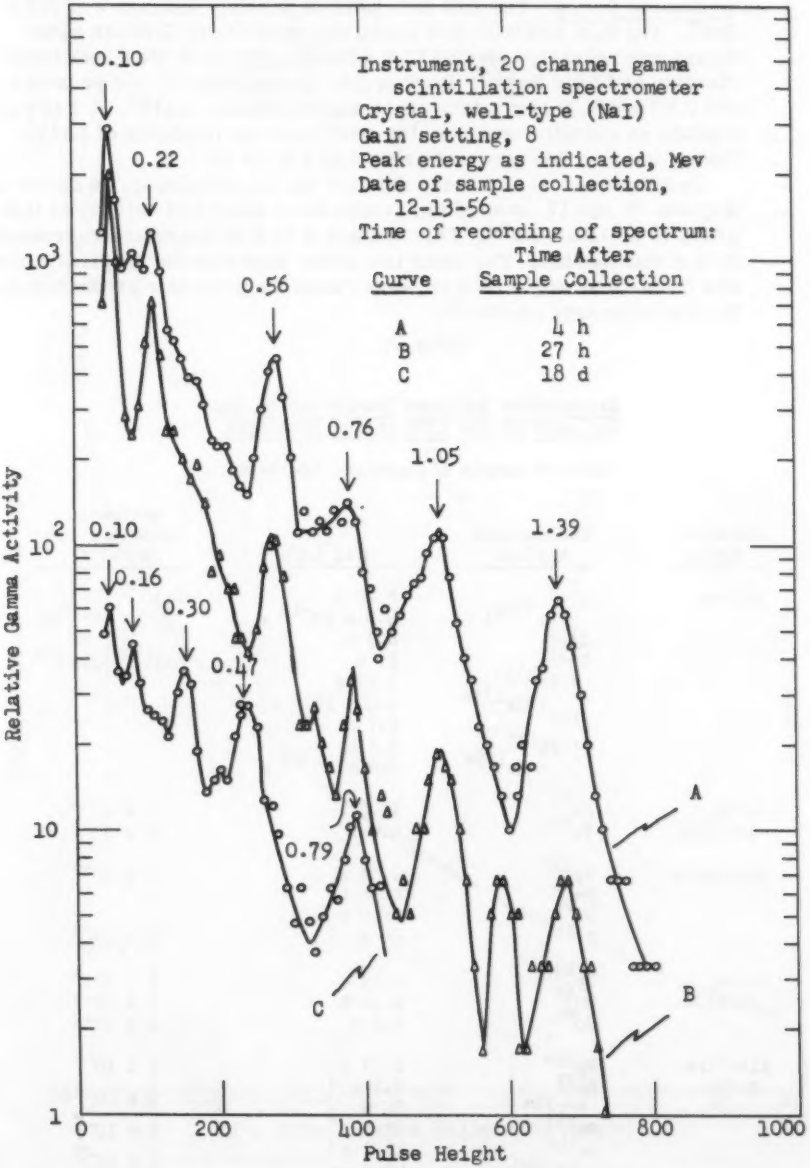

Fig. 14. Gamma Spectral Data for Influent to LITR Cation Exchanger: Alkaline Earth Group.

peaks showed them to be associated with 9.7 h Sr^{91} + Y^{91m}. As was mentioned earlier, evidence for 50 m Y^{91m} had been found in the Hydroxide Group. The 1.39 Mev peak is possibly indicative of 2.7 h Sr^{92}. The 0.10 and 0.30 Mev peaks shown in Curve C of the same figure are representative of 27.4 d Pa^{233}. The 0.16 Mev peak is indicative of 12.8 d Ba^{140}. Barium-140 is confirmed by the presence of the 0.47 and 0.79 Mev peaks of its daughter 40.2 h La^{140}. A LaF_3 precipitate of a similar sample also confirmed the presence of La^{140}. Decay data for this group are shown in Figure 15.

Gamma spectral and decay data for the Soluble Group, as shown in Figures 16 and 17, showed the predominant source of activity in this group to be 15.0 h Na^{24}. The 1.34 and 2.75 Mev peaks are representative of this nuclide. The other two peaks shown on the spectral curve are double and single escape peaks resulting from pair production by the higher energy photon.(4)

TABLE II

Radioactive Nuclides Identified in the Influent to the LITR Cation Exchanger

Date of sample collection, 12-13-56

Chemical Group	Radioactive Nuclide	Half Life	Estimated Concentration (μc/ml)*
Silver	I^{131}	8.05 d	8×10^{-5}
	+ Xe^{131m}	4.8×10^{-10} s	
	I^{132}	2.3 h	
	I^{133}	21 h	6×10^{-4}
	+ Xe^{133}	5.27 d	
	+ Cs^{133m}	6.0×10^{-9} s	
	I^{135}	6.7 h	
	+ Xe^{135}	9.2 h	
	+ Cs^{135m}	2.8×10^{-10} s	3×10^{-3}
Acid Sulfide	Cu^{64}	12.8 h	1×10^{-3}
	Tc^{99m}	6.0 h	7×10^{-4}
Hydroxide	Np^{239}	2.33 d	5×10^{-4}
	Mn^{56}	2.58 h	
	Na^{24}	15.0 h	
	Y^{91m}	50 m	1×10^{-4}
Basic Sulfide	Np^{239}	2.33 d	1×10^{-4}
	Mn^{56}	2.58 h	7×10^{-4}
	Co^{60}	5.2 y	1×10^{-5}
Alkaline Earth	Np^{239}	2.33 d	2×10^{-4}
	Sr^{91}	9.7 h	5×10^{-4}
	+ Y^{91m}	50 m	
	Pa^{233}	27.4 d	1×10^{-5}
	Ba^{140}	12.8 d	1×10^{-5}
	+ La^{140}	40.2 h	
Soluble	Na^{24}	15.0 h	3×10^{-3}

* At time of sample collection.

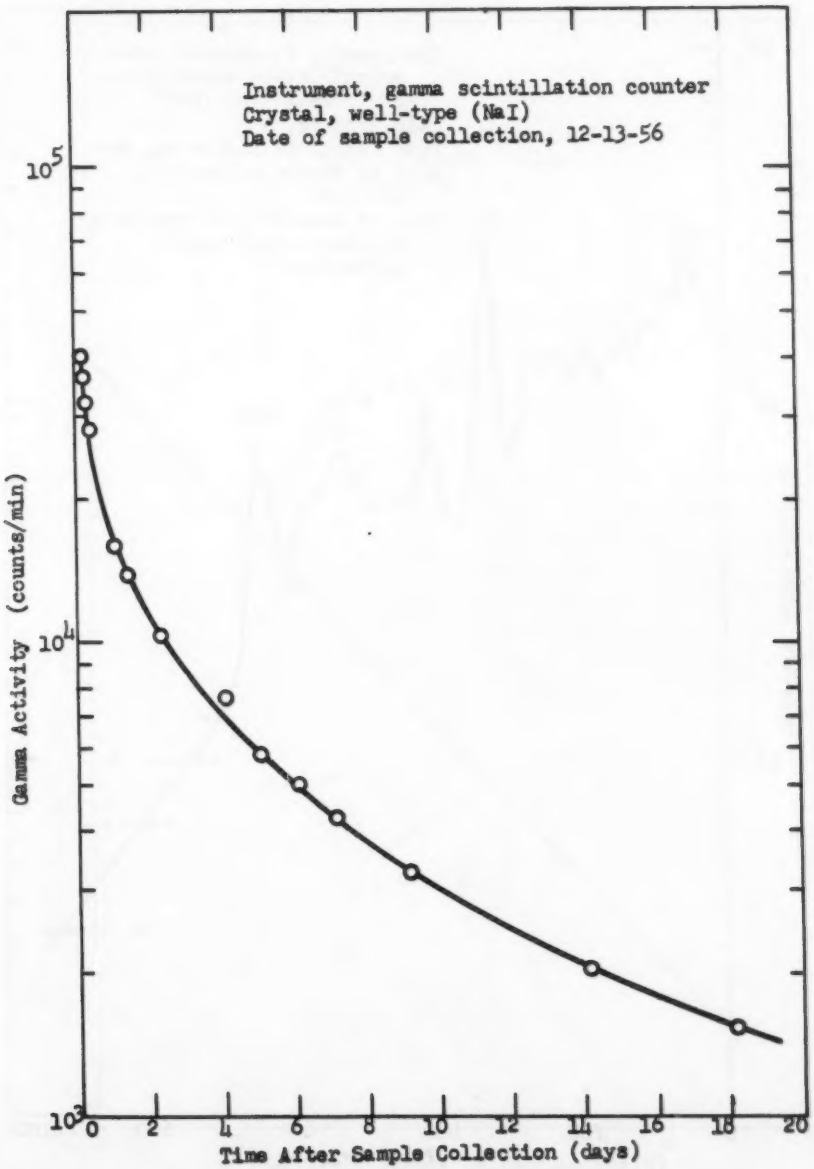

Fig. 15. Gross Gamma Decay for Influent to LITR Cation Exchanger: Alkaline Earth Group.

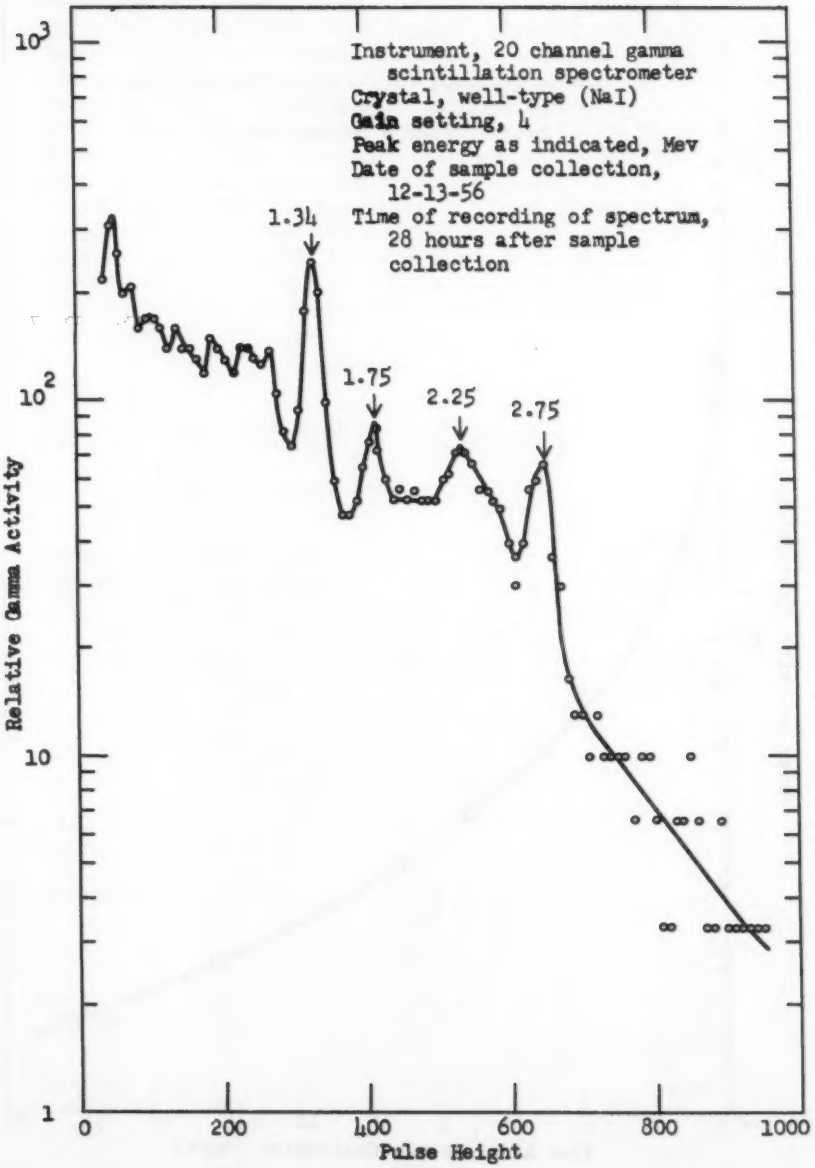

Fig. 16. Gamma Spectral Data for Influent to LITR Cation Exchanger: Soluble Group.

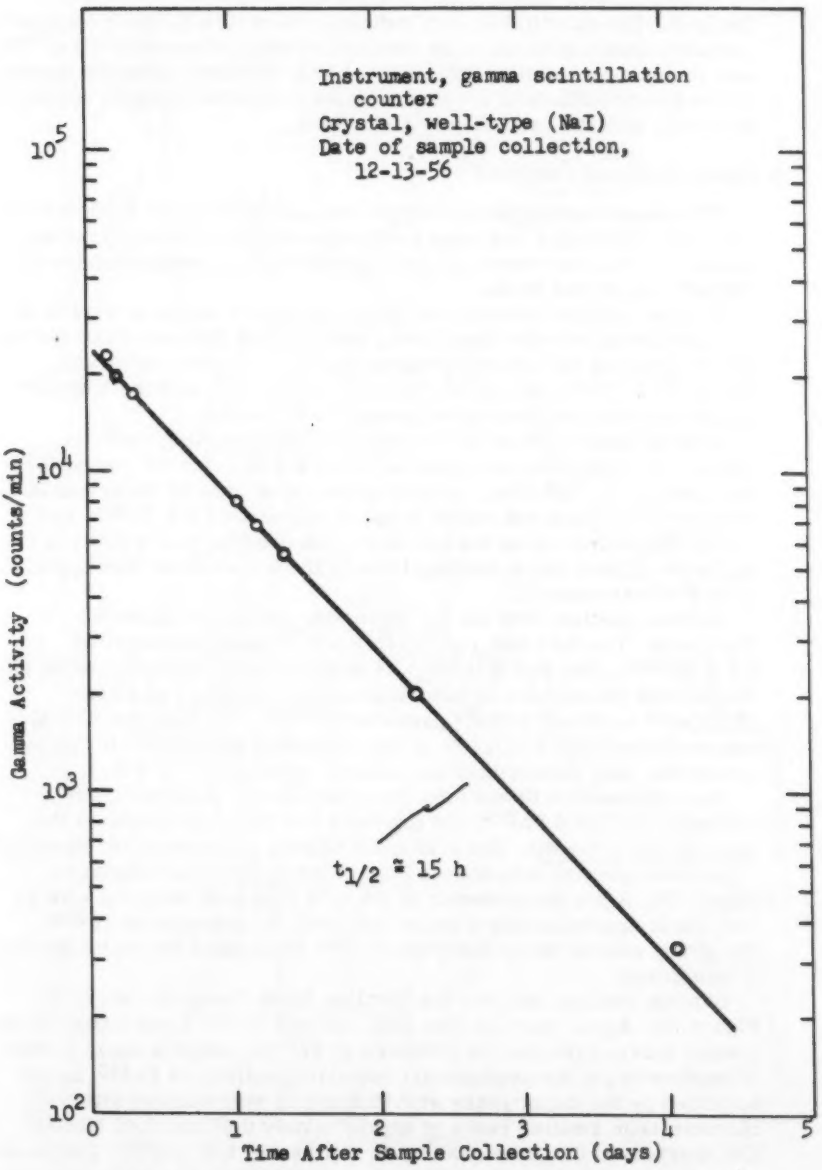

Fig. 17. Gross Gamma Decay for Influent to LITR Cation Exchanger: Soluble Group.

A summary of radioactive nuclides identified in the cation exchanger influent and an estimate of their concentrations are given in Table II. The quantitative data indicate orders of magnitude only, with calculations being based on an assumed average chemical yield of 75% and on a gamma counting efficiency of 33%. However, specific quantitative determinations of several nuclides in similar samples compared favorably with the estimates reported here.

B. Cation Exchanger Effluent

The second cooling water sample was collected at the effluent from the cation exchanger and chemically separated as previously outlined. Because of the lower over-all activity expected, a sample volume of 280 milliliters was taken.

Gamma spectral data for the Silver Group are shown in Figure 18. The similarity between these decay and spectral data and those for the Silver Group of the cation exchanger influent is readily apparent. Again, 8.05 d I^{131}, 2.3 h I^{132}, 21 h I^{133}, 6.7 h I^{135}, and their daughter products were concluded to be present in the sample.

Gamma spectral data for the Acid Sulfide Group are shown in Figure 19. Two prominent gamma peaks are in evidence, one at 0.14 Mev and one at 0.57 Mev. A study of the gross gamma decay and the chemistry of this group reveal these to represent 6.0 h Tc^{99m} and 2.8 Sb^{122}. Inasmuch as the 0.14 Mev peak decayed with a half life of six hours, it was concluded that little of the longer lived Mo^{99} parent of Tc^{99m} was present.

Gamma spectral data for the Hydroxide Group are shown in Figure 20. The 0.14 Mev peak in Curve A is again indicative of 6.0 h Tc^{99m}. The fact that the 0.14 peak was still noticeable after the sample had decayed for 10 days suggests the presence of either 32 d Ce^{141} or the 77 h Mo^{99} parent of Tc^{99m}. The 0.51 and 1.02 Mev peaks decayed with a half life of approximately two hours. It was concluded that they represented the positron emitter 1.87 h F^{18}.

The prominent 0.10 and 0.30 Mev peaks shown in Curve C are indicative of 27.4 d Pa^{233}. Its presence was later confirmed in the Alkaline Earth Group. The 0.23 and 0.67 Mev peaks were not identified.

Gamma spectral data for the Basic Sulfide Group are shown in Figure 21. Again the presence of the 0.14 Mev peak decaying with a half life of approximately 6 hours indicates the presence of Tc^{99m}. The gross gamma decay indicated Tc^{99m} constituted the major portion of this group.

Gamma spectral data for the Alkaline Earth Group are shown in Figure 22. Again, the 0.14 Mev peak, as well as the rapid initial gross gamma decay, indicates the presence of Tc^{99m}. After a decay period of fourteen days, the predominant remaining activity is Pa^{233}, as exemplified by the major peaks at 0.10 and 0.31 Mev coupled with its characteristic smaller peaks at approximately 0.18 and 0.40 Mev.(5) The short-lived 0.51 Mev peak again represents 1.87 h F^{18}. The presence of the shoulder on the 0.51 Mev peak indicates that perhaps some Sr^{91} + Y^{91m} remains in the cooling water.

The Soluble Group contained an insignificant amount of radioactive material.

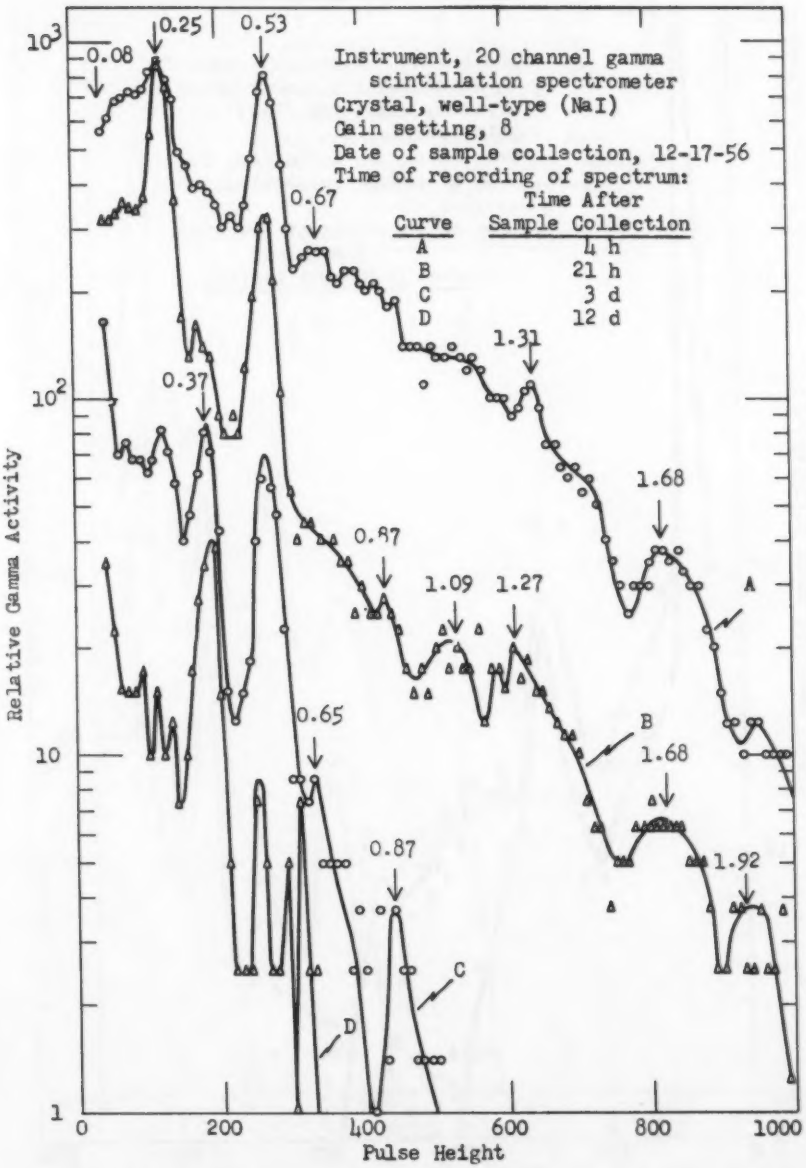

Fig. 18. Gamma Spectral Data for Effluent from LITR Cation Exchanger: Silver Group.

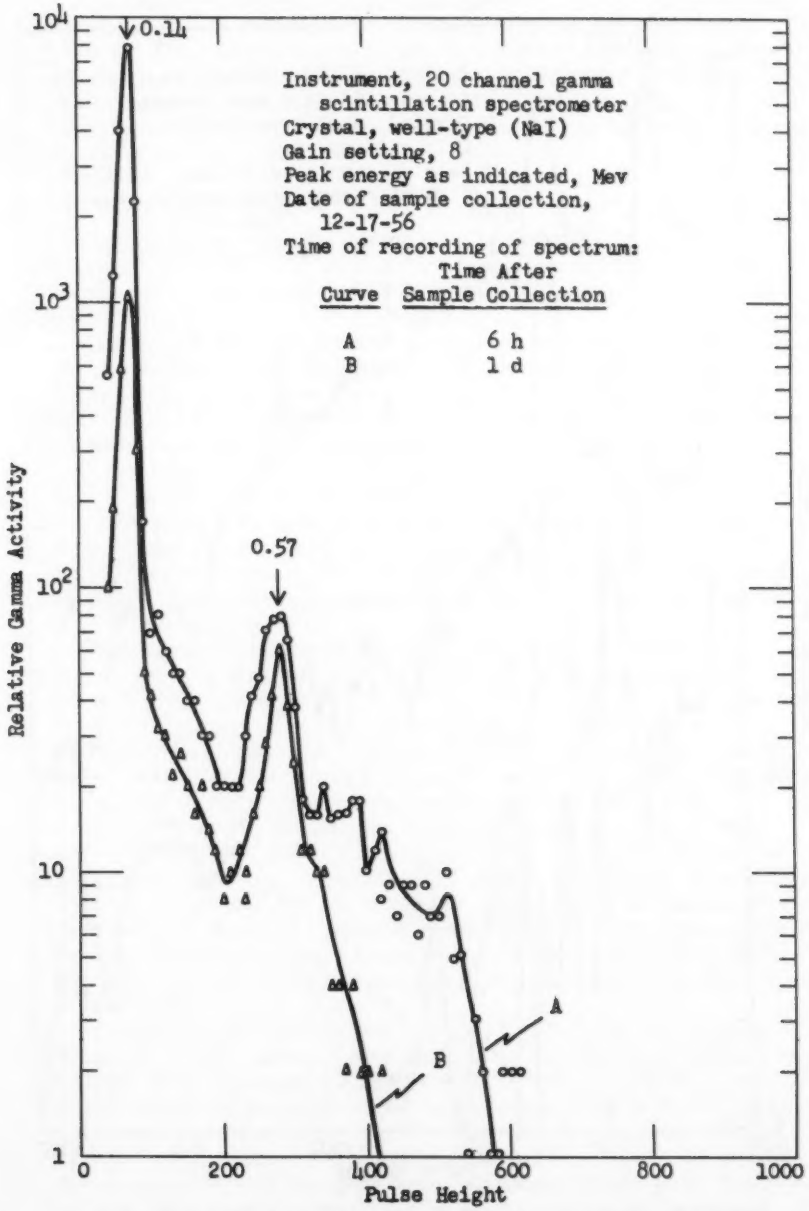

Fig. 19. Gamma Spectral Data for Effluent from LITR Cation Exchanger: Acid Sulfide Group.

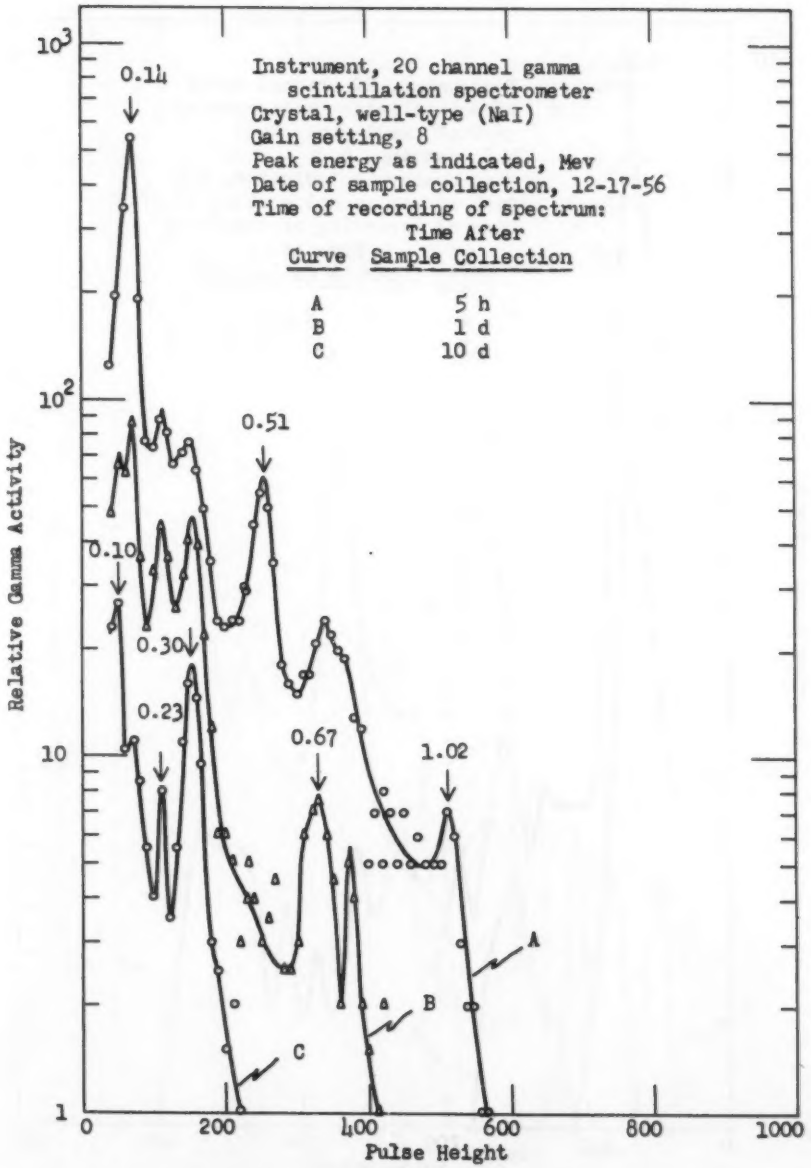

Fig. 20. Gamma Spectral Data for Effluent from LITR Cation Exchanger: Hydroxide Group.

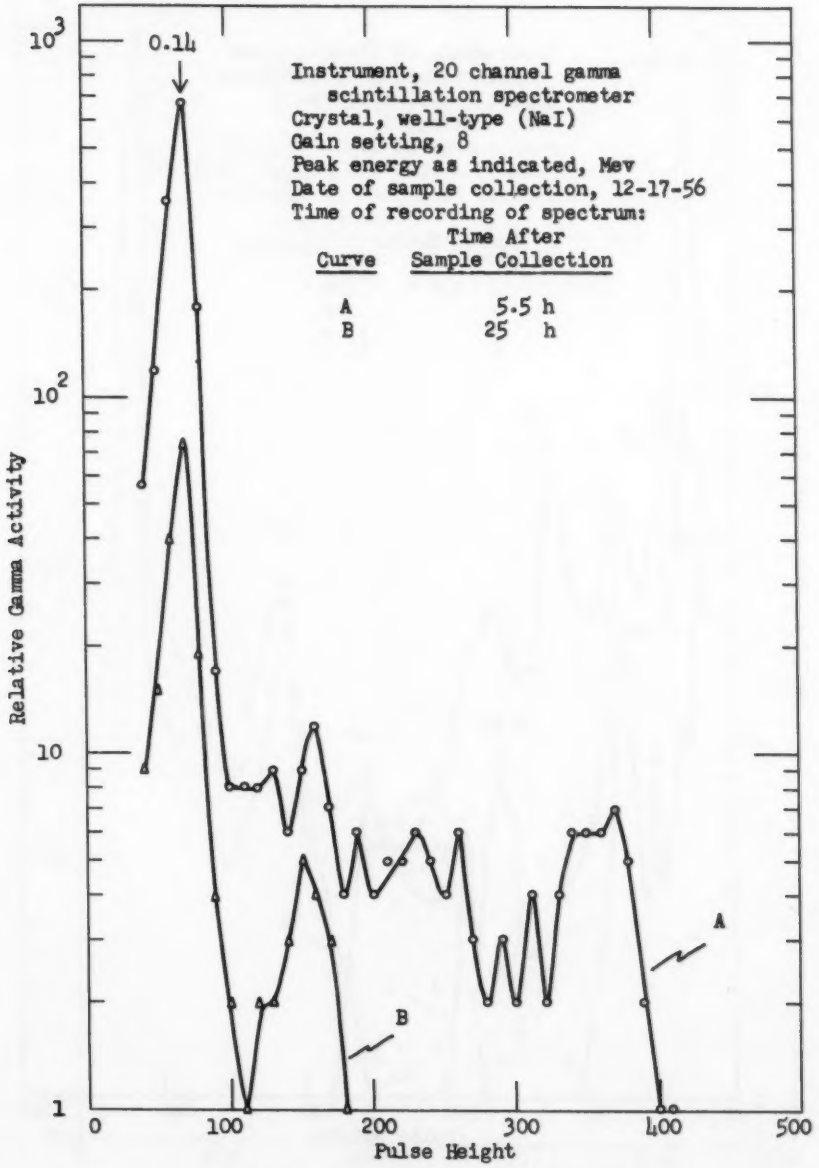

Fig. 21. Gamma Spectral Data for Effluent from LITR Cation Exchanger: Basic Sulfide Group.

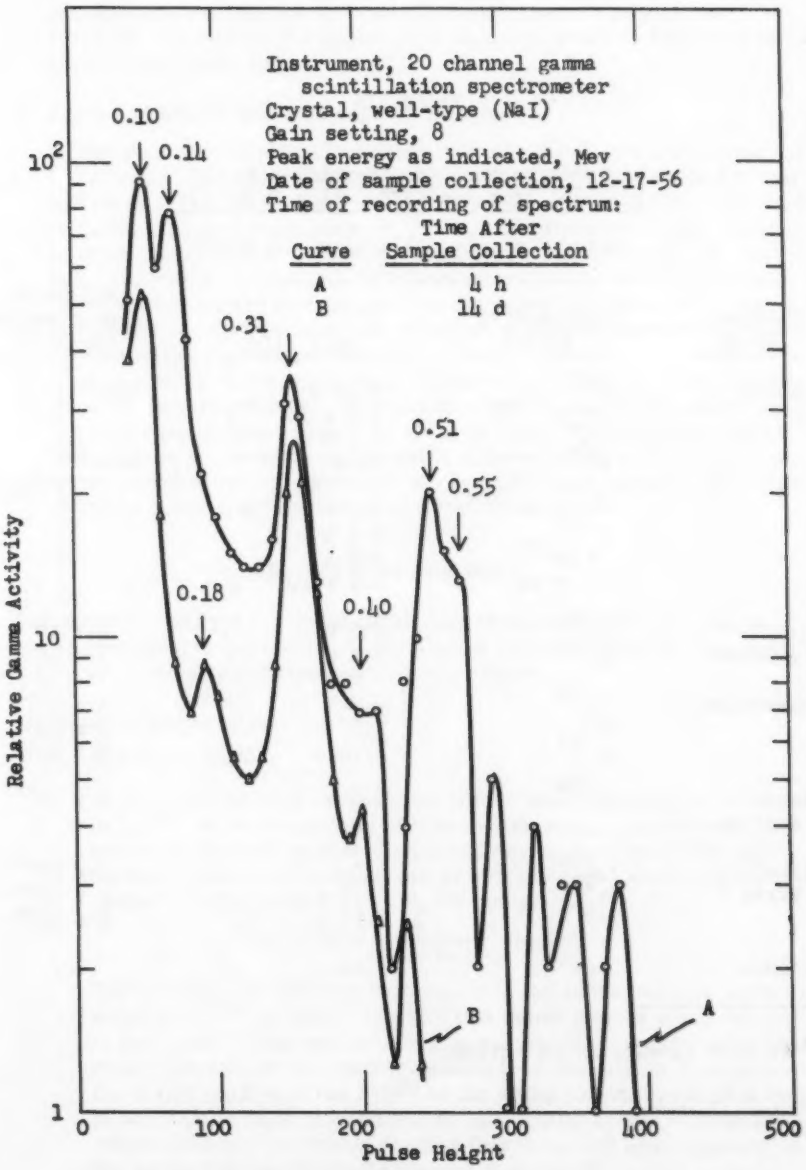

Fig. 22. Gamma Spectral Data for Effluent from LITR Cation Exchanger: Alkaline Earth Group.

TABLE III

Radioactive Nuclides Identified in the Effluent from the LITR Cation Exchanger

Date of sample collection, 12-17-56

Chemical Group	Radioactive Nuclide	Half Life	Estimated Concentration (μc/ml)*
Silver	I^{131}	8.05 d	4×10^{-5}
	+ Xe^{131m}	4.8×10^{-10} s	
	I^{132}	2.3 h	
	I^{133}	21 h	4×10^{-4}
	+ Xe^{133}	5.27 d	
	+ Cs^{133m}	6.0×10^{-9} s	2×10^{-3}
	I^{135}	6.7 h	
	+ Xe^{135}	9.2 h	
	+ Cs^{135m}	2.8×10^{-10} s	
Acid Sulfide	Tc^{99m}	6.0 h	4×10^{-4}
	Sb^{122}	2.8 d	2×10^{-5}
Hydroxide	Tc^{99m}	6.0 h	1×10^{-4}
	F^{18}	1.87 h	5×10^{-5}
	Pa^{233}	27.4 d	5×10^{-6}
Basic Sulfide	Tc^{99m}	6.0 h	5×10^{-5}
Alkaline Earth	Tc^{99m}	6.0 h	5×10^{-6}
	Pa^{233}	27.4 d	4×10^{-6}
	F^{18}	1.87 h	5×10^{-6}
Soluble	---	---	---

* At time of sample collection.

A summary of radioactive nuclides identified in the cation exchanger effluent and of their relative concentrations is given in Table III. As before, the quantitative data are meant to illustrate orders of magnitude only.

C. Anion and Mixed-Bed Exchanger Effluents

Because of reduced radionuclide concentrations, sample volumes of two liters were used in studies of the effluents of the anion and mixed-bed exchangers. In general, the chemical separation scheme previously outlined proved inadequate for positive identification of the nuclides in these two samples. However, it did prove useful for making assumptions regarding the nuclides and these assumptions were then either confirmed or denied through specific chemical separations for the particular elements in question, followed by gamma spectral and decay studies of the separated fractions.

A summary of the nuclides identified in these two samples as well as those identified in the coolant at the other stages of the LITR cooling water purification system is given in Table IV. Also included in this table is an estimate of the stable element content of the make-up water added to replace losses in the system. This estimate was made utilizing the neutron activation analysis technique.(6)

Source of Radionuclides

As may be seen by a review of the data presented in Table IV, radionuclides identified in the cooling water system of the LITR were generally of two groups: induced activities and fission products.

A. Induced Radionuclides

1. Np^{239} and Pa^{233}

Perhaps the first radionuclide whose source should be discussed is Np^{239}. A large quantity of this nuclide would certainly not be expected in the cooling system of a reactor such as the LITR, where the fuel enrichment is in excess of 90% U^{235}, the most common method for the production of Np^{239} being:

$$U^{238}(n,\gamma)U^{239} \xrightarrow[23.5\ m]{\beta} Np^{239}$$

The presence of Np^{239} in the coolant of the LITR has been known for some time.(7) In view of the fact that known breaks occurred several years ago in the exposure of natural uranium samples within the coolant stream of the reactor vessel, it is reasonable to assume that the major portion of the Np^{239} in the water has occurred as a result of natural uranium contamination. Attribution of Np^{239} to this source is substantiated by the fact that it has not been reported in the coolant of the Materials Testing Reactor.(3)

Similarly, Pa^{233} probably originates from thorium contamination within the reactor vessel, there having been ample opportunity for such contamination during many tests conducted within the system in the past.

TABLE IV

Summary of Radioactive Nuclides Identified in Water at Various Stages of LITR Cooling Water Purification System

Estimated concentrations given in units of 10^{-5} μc/ml

Make-Up Water		Cation Exchanger Influent		Cation Exchanger Effluent		Anion Exchanger Effluent		Mixed-Bed Exchanger Effluent	
Radionuclide	Est. Conc.*	Radionuclide	Est. Conc.**	Radionuclide	Est. Conc.**	Radionuclide	Est. Conc.**	Radionuclide	Est. Conc.**
Na^{24}	500	I^{131}, I^{132}, I^{133}, I^{135}	300	I^{131}, I^{132}, I^{133}, I^{135}	200	I^{132}, I^{133}	3	Cu^{64}	1
Br^{80}, Br^{80m}, Br^{82}	80	Na^{24}	300	Tc^{99m}	50	Cu^{64}	1	Ce^{143} + Pr^{143}	0.4
F^{18}	30	Cu^{64}	100	F^{18}	5	Te^{132} + I^{132}	1	Mo^{99} + Tc^{99m}	0.3
Mn^{56}	10	Np^{239}	80	Sb^{122}	2	Mo^{99} + Tc^{99m}	0.3	Te^{132} + I^{132}	0.2
Cu^{64}	4	Tc^{99m}	70	Pa^{233}	1	Ce^{143} + Pr^{143}	0.3	Ce^{141}	0.1
		Mn^{56}	70			Sb^{122}	0.2	Cr^{51}	0.1
		Sr^{91} + Y^{91m}	50			Ce^{141}	0.1	I^{131}, I^{132}, I^{133}, I^{135}	<0.01
		Pa^{233}	1			Cr^{51}	0.1		
		Co^{60}	1			Fe^{59}	0.05		
		Ba^{140} + La^{140}	1			I^{131}	0.03		

* At end of 16 hour irradiation at flux of 6.5×10^{11} neutrons·cm^{-2}·sec^{-1}.

** At time of sample collection.

2. Na^{24}

Of the remaining radionuclides thought to originate from induced activity, Na^{24} is of the most importance. Relatively large quantities have been observed in the LITR cooling system since initial operation. Early studies by Binford and Lyon indicated that this radionuclide came from sources other than stable elements in the water.(8) Quantitative neutron activation analysis, during this study, of the demineralized make-up water used in the LITR cooling system showed it to contain an average of about 10^{-2} ppm stable sodium. Using the average thermal neutron flux and retention time of the water in the reactor core, an estimate can be made of the amount of Na^{24} which would result from 10^{-2} ppm of stable sodium in the circulating coolant. Such estimates reveal that the stable sodium content of the water would account for only one hundredth to one tenth of the Na^{24} routinely found in the water, thus supporting the conclusion made by Binford and Lyon.

Heath,(3) through studies of the cooling water system of the MTR, concluded that the major source of Na^{24} in that reactor was the $Al^{27}(n,\alpha)Na^{24}$ reaction. It is quite probable that aluminum is the principal source of Na^{24} in the LITR also.

3. F^{18}

Inasmuch as F^{18} would not be produced in neutron irradiated water by the common (n,γ) reaction, some time was devoted to determining its source. Addition of stable fluoride in the form of hydrofluoric acid to similar water samples prior to irradiation showed the source of the F^{18} was not stable fluorine, i.e., by the reaction $F^{19}(n,2n)F^{18}$. After additional study, it was finally concluded that the most likely source of the F^{18} was the (p,n) reaction on O^{18}, the source of the protons being "knock-on" protons produced from the water by fast fission neutrons. The cross section for the $O^{18}(p,n)F^{18}$ reaction for protons of 4 Mev energy has been reported to be 0.2 barns, the threshold for the reaction being 2.56 Mev with a reasonance maximum at 3.55 Mev.(9) Heath(3) had previously reported the finding of F^{18} in MTR cooling water and had reached the same conclusion as to its source.

4. Cu^{64}, Mo^{99}, Co^{60}, Mn^{56}, Sb^{122}, Cr^{51} and Fe^{59}

Most of these radionuclides probably originate from neutron irradiation of stable elements in the make-up water, itself, and in the structural material within the reactor system. The water in the cooling system is exposed to large amounts of steel surfaces in the reactor tanks, valves, pump housing, fittings, and a portion of the pipe. In addition, the demineralizer system, the pump shafts, and the impellers are of stainless steel, making a total area of exposed steel in the system of about 700 square feet. Common impurities in steel include molybdenum, manganese, and chromium.

B. Fission Products

Fission products make up the second group of radionuclides whose

source should be discussed. Initially, it was thought that the main source of these nuclides might be gaseous diffusion through the fuel element cladding, inasmuch as the majority of them are descendents of fission product gases.

Two isotopes of cesium are prominent fission products. One of these, 30 y Cs^{137}, is a descendent of the fission product gas Xe^{137}. The other isotope, 13 d Cs^{136}, however, is formed directly in fission. A check on the presence of these two isotopes in the cooling water system should furnish data either supporting or disproving the gaseous diffusion theory.

Cesium for the study was obtained from a sample of the cation resin. Both isotopes of cesium were found to be present. Therefore, it was concluded that the cooling water contained fission products directly yielded in fission as well as those descended from fission product gases.

Presence of the short-lived isotopes of iodine in the water substantiated the fact that the fission products reach the water shortly after formation. It was not probable that they came from enriched uranium contamination of the reactor vessel, since no enriched samples had been exposed within the coolant stream and no known fuel element ruptures had occurred. These and other facts tend to point to the standard fuel elements themselves as the source.

Neutron activation analysis[6] of aluminum scraped from the cladding of a typical new enriched uranium fuel plate showed it to be contaminated with U^{235}. The degree of contamination was adequate to account quantitatively for the fission products in the coolant. Therefore, it was concluded that such contamination constituted the principal source of these radionuclides.

SUMMARY AND CONCLUSIONS

Cooling systems are dynamic, being influenced by many variables and constantly changing as these influences have their effect. Data regarding the nature and levels of radionuclides in cooling water reflect unusual conditions in the reactor system. Such conditions may include fuel element rupture, failure of experimental facilities, and excessive corrosion. As a result, routine monitoring of the coolant is a necessity for safe, efficient reactor operation.

It has been shown that there was adequate uranium contamination on or near the surface of the aluminum cladding of typical new fuel plates to account quantitatively for the fission products in the coolant. Such contamination probably occurs during fabrication of the elements. Secondary sources of fission products may arise through pitting of fuel plates and spills of experimental uranium samples within the reactor vessel.

The induced activities were thought to originate through neutron activation of reactor structural materials and of impurities in the make-up water. Vaious nuclides in the structural materials could gain access to the water, either before or after activation, through corrosion processes. It has been shown that the ion exchange system employed in the LITR is effective in removing radionuclides from the cooling water. Maintenance of high purity water in the reactor system has had many advantages. These include lowered

radiation intensities surrounding the cooling water pumps and piping, reduced contamination on tools used within the reactor tank, and reduced accessibility problems. In addition, the use of water of high specific resistance has resulted in much lower corrosion rates --- an important factor in the life of the reactor.(7)

The effectiveness of the control methods in the LITR cooling system eliminates the need for the release of radioactive material to the environment through cooling water discharge. Radioactive material removed from the water is retained on the ion exchange resins until removed by regeneration. The small volume of regenerating solutions is adaptable to storage. Conversely, storage of the large volumes of water discharged from reactors employing the single pass system is impractical. In addition, there is the possibility that significant amounts of radioactive material could be discharged into the environment from single pass systems, should fuel element rupture or other accidents release radionuclides into the coolant stream. Therefore, from the public health point of view, the recirculating system is the type to be recommended.

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THE HYDROLOGY OF URBAN RUNOFF[a]

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SYNOPSIS

Presented in this paper is a detailed study of rainfall-runoff relationships in urban areas based upon a "Design Storm" of three hours duration. The "Chicago Hydrograph Method" of sewer design evaluates, in detail, the rainfall abstractions and flow detentions which intervene between the hyetograph of rainfall and the hydrographs of sewer supply and sewer outflow.

Several types of uniform land use with various values of ground slope and depression pondage have been studied.

Based upon the sewer hydrographs, thus derived, a series of "easy-to-use" design charts are presented, that can later be expanded to cover other combinations of the interdependent variables. The use of electronic computers has made such calculations feasible.

For economical trunk sewer design in drainage areas, of a square mile or more, the authors present a further study of the hydrographs of sewer flow to delineate the surface profiles of instantaneous backwater-drawdown curves.

An evaluation of the effect of nonuniform areal distribution of rainfall upon the sewer flow hydrographs in large drainage areas is also presented.

A sewer gauging station, coupled with a recording rain gauge is being established in a fully developed, ten acre, residential area. The Chicago sewer pattern includes approximately ten thousand such "unit" areas, with various Types of Land Use.

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Notation

The letter symbols introduced in this paper are defined where they first appear, in the text, or by illustration, and are assembled alphabetically in the Appendix for convenience or reference.

INTRODUCTION

Immediately following World War II, the City of Chicago embarked upon the largest sewer construction program in its history. An engineering report prepared in 1944 by the Bureau of Sewers had indicated the urgent need for $86,000,000 worth of new sewer construction. Further studies and the continued growth of the city have now raised that figure for the total program to $165,000,000. A new sewer planning division was organized to prepare plans and specifications. Bond issues totalling 96 million dollars have been approved by the voters. Additional bond issues are planned.

The "Rational Method" for determination of quantities of stormwater runoff had been used by the city for several decades. This method which appeared first about the year 1900 is still the most widely accepted method in America. With a program of sewer construction involving more than $100,000,000 of expenditures, it seemed proper to re-examine the validity of the "Rational Method" as currently applied to the problems of urban runoff and to explore the possibilities of a new approach to the understanding and solution of such problems.

Many books have been published dealing with the hydrology of stream flow in mixed urban and rural areas. In large drainage areas involving river systems, the concept of hydrographs has become the basic approach. Some studies of the application of hydrographs to the problems of urban drainage have been made but none of these have been sufficiently developed to produce an easy-to-use, yet comprehensive method for sewer design.

Sewers as in Chicago, with widths as great as 20 feet, draining thousands of acres of highly developed urban area are essentially underground rivers and the concepts of the river hydrologists seem applicable. In Chicago, therefore, it became desirable to attempt an understanding of the time of occurrence vs. rate-of-flow relationship at principal points in the trunk lines during actual or hypothetical rainstorms. If these relationships could be established in the form of sewer flow hydrographs, instantaneous backwater or drawdown curves could be computed therefrom and the performance of the trunk sewer more adequately understood.

The use of the Rational Method without modification is, without doubt, the simplest means of obtaining an answer to the rainfall runoff relationship. The formula is usually expressed as $Q = Ci_{ave}A$, where "A" is defined as the area in acres tributary to the point of design. "i_{ave}" is defined as the average intensity of precipitation for a duration equal to the time of flow from the most remote portion of the drainage area to the point of design. "i_{ave}" must be selected from rainfall statistics related to the probable frequency of occurrence of storms of such intensities. "Q" is rate of runoff expressed in cubic feet per second.

The remaining component of the equation is "C" which is a complex variable having concealed within it numerous interdependent variables such as the character of land use, the extent of land coverage by impervious

surface, the slope of ground surface, the infiltration capacity of the pervious area, the cumulative volume of minor or major puddles and pools, the length of overland flow and the roughness coefficient of all overland flow surfaces and channels. Where drainage areas are now only partially developed, a future stage of development must be assumed. The compositing of all of these interdependent variables in their proper proportion to develop the value of "C" cannot be methodically conceived by any form of mental exercise. Consequently, the determination of "C" must come out of years of experience in a particular locality and cannot be adequately nor easily conveyed to a novice. Too often, due to economic necessity, the value of "C," being only adjustable value in the formula is, therefore, adjusted to match the available dollars.

In the following presentation, various types of uniform land use, ranging from suburban residential to industrial and commercial, have been analyzed and the end results determined. The principal influences which have been evaluated for each type of land use are:

1. Infiltration Capacity of Pervious Areas
2. Depression Storage
3. Overland Flow Detention
4. Detention in Gutters
5. Detention in Lateral Sewer Systems

Having developed a methodology and derived or assumed the numerical values of these peak-reducing factors, each of these factors can be subject to discussion and review by other investigators, and such efforts will clarify the concepts of rainfall-runoff relationships in urban areas.

The unit of 10 acres is typical of the Chicago street pattern and may be typical of some other cities. There are more than 10,000 such lateral sewers in the city of Chicago. At any rate, the computed outflow hydrographs from a 10-acre drainage area, with the rate of runoff expressed in inches per hour, would be directly comparable with similar hydrographs for areas of approximately the same size and land use in other cities having similar rainfall.

A condensed description of the hydrograph type of analysis as used by the City of Chicago is outlined in the following pages. The steps followed in the Chicago methodology can be stated briefly as follows:

Step 1. Determination of the typical layout of the drainage areas.

Step 2. Determination of chronological storm pattern or hyetograph of a Design Storm.

Step 3. The evaluation of the abstractions from rainfall.

Step 4. The determination of runoff hydrographs at the lower end of elemental strips of both pervious and impervious ground surfaces and roofs.

Step 5. Routing of the mixed flow from the elemental strips of pervious and impervious ground surfaces along the channel of the street gutter to the inlet or catch basin grating.

Step 6. Routing of the sewer supply hydrographs from roofs and street inlets along a typical headwater sewer lateral.

Step 7. Routing of the lateral outflow hydrograph by a time-offset method

along the sub-mains and main outlet sewers to the point of discharge of the sewer system.

Step 8. The production of a series of easy-to-use design charts.

Each of the above steps are described in some detail in this paper with illustrative computations, followed by two illustrative problems using the charts produced under Step 8. One is for a small drainage area and the other is for a large drainage area showing the instantaneous backwater profiles, which are necessary for the understanding of flow conditions in large trunk sewers and their major branches.

The intricacy and volume of computations required for the derivation of the final series of charts would have been too formidable without the introduction of the electronic computer. The basic concepts have been adjusted slightly and the graphical relationships have been translated into mathematical formulae in order to utilize the timesaving value of the electronic computer. Such adjustments have been checked as to their effect by comparison at critical points with results obtained by the manual computations.

The steps numbered and named in this introduction are used as section headings in succeeding pages of this paper.

Step 1 - Street Layout and Land Use

A determination must be made of the layout of the basic drainage area. The path that the water must take to reach the underground sewer system, the length and slope of overland flow strips, the location of gutter summits, the length and slope of gutters, the building arrangement, whether the roofs are directly connected to the underground sewers or discharge through the downspouts on to lawn areas, the roughness factor to be used on all surfaces, and other physical measurements must be set before computation can proceed.

For the City of Chicago, in general, a system of parallel main sewers are laid out at half-mile intervals. Each of these mains have a set of lateral sewers extending for a quarter-mile on each side of the main sewer, see Figure (1). It was decided to use the 10-acre catchment area of the laterals as the unit area to be studied. This area consists of two standard five acre blocks, each 330 feet wide by 660 feet in length.

Figure (2) shows the drainage plan of a typical ten-acre unit area using Type 5 Land Use as an example. The upper plan shows the lot dimensions and arrangement, the street lines, curb lines, and the location of catch basins and manholes. The lower plan, on the left hand side, gives the tributary areas of each catch basin, and the arrows show the direction of flow. On the right hand side, the catchment areas are broken down into pervious and impervious portions, with the impervious areas crosshatched. The areas in acres are given for each of these sub-areas. Also shown, are the elemental strips for which overland flows are computed, and a table below for description, length, slope, and Izzard's C for each of these strips.

Several simplifying assumptions are introduced in the layout of these blocks, namely:

1. Public and private sidewalks drainage on to pervious areas are herein considered a part of the pervious area. This is at least partially justified by the fact that the infiltration capacity of the adjacent areas

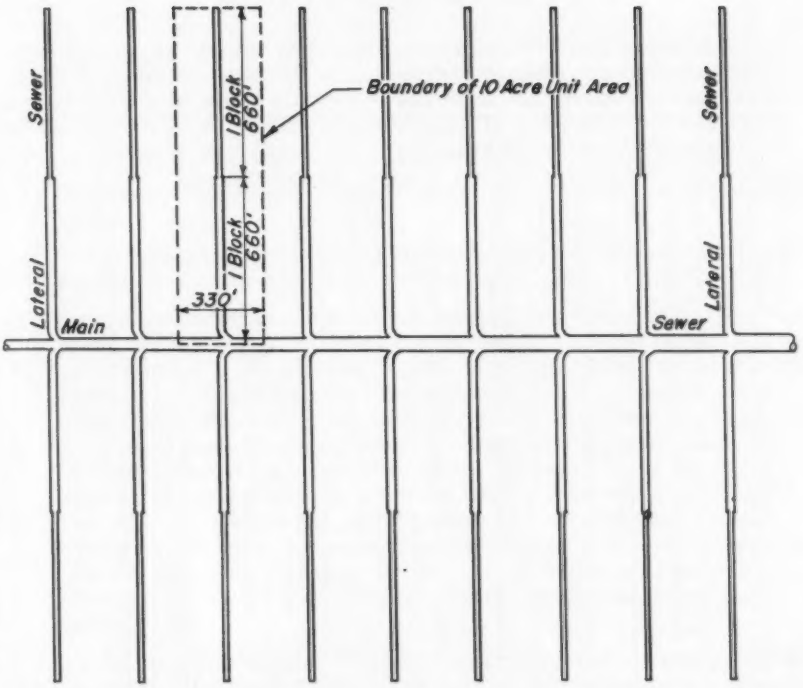


Fig. 1 – TYPICAL LAYOUT OF LATERAL AND MAIN SEWERS

will be increased due to the lateral spread of water under the walks. Depression storage may also be increased if the lawns are not graded even with the walks.

2. Building roofs are converted into a uniform strip of equivalent area. This provides an average length of overland flow, both to the front gutter and in the rear, to the alley gutter.
3. The garage roofs in this type of land use, since they discharge onto backyard lawns, were considered a part of the pervious areas. For other types of land use, part or all of the garage roofs were added directly to the alley gutter.

Other unit areas with various uniform land use have been established and were divided into sub-areas similar to that illustrated with Type 5 Land Use.

The runoff characteristic and outflow hydrographs from the ten acre unit area will vary with the type of land use, infiltration capacity, depression storage, surface slope and other factors. These are discussed in later sections of this paper.

PLAN SHOWING LAND USE AND SEWERAGE
PLAN SHOWING DRAINAGE AREA AND DIRECTION OF FLOW
CATCHMENT AREAS
DIRECTLY CONNECTED IMPERVIOUS PORTIONS
TOTAL IMPERVIOUS AREAS

Fig 2 - DRAINAGE PLAN OF A TYPICAL TWO-BLOCK UNIT AREA, WITH TYPE 5 LAND USE

Step 2 - Design Storm Pattern

The hydrograph method of sewer design must have as a prerequisite, a rainfall hyetograph of specific shape and size in order to chronologically subtract the losses due to infiltration and surface depression storage to obtain the hydrograph of surface runoff supply. In order to select this hyetograph or design storm pattern, the following steps must be made:

1. Selection of allowable average frequency of overloading of the proposed sewer.
2. Determination, from rainfall statistics, of the intensity-duration curve for the selected frequency of overloading.
3. Determination of the chronological storm pattern or hyetograph from those storms which are most likely to cause excessive runoff. A detailed study of a 100 or more storms occurring in the last twenty-five years in the City of Chicago have indicated that the summer thunderstorm can be considered the critical type storm for that locality. An analysis of these excessive storms for antecedent precipitation preceding various rainfall durations having maximum mass rainfall for such durations and an analysis of the time of occurrence of the peak rate of rainfall within the storm period, will classify the storms as having an advanced or delayed peak with regard to the mid-point of the storm's duration. The majority of these storm patterns have an advanced peak located at about three-eighths of the total duration of storm.

Using the peak location as determined under (3), and the intensity-duration curve selected under (2), a design storm pattern or hyetograph can be computed so as to conform at all points to the intensity-duration curve. The three-hour duration of the design rainstorm was selected to cover the time of travel from headwater to outlet in the largest individual sewer systems within the City of Chicago.

The equation for the Design Storm Pattern as developed in Reference (1) is,

$$\text{(Before The Peak)} \quad i = \frac{a\left[(1-b)\left(\frac{t_b}{r}\right)^b + C\right]}{\left[\left(\frac{t_b}{r}\right)^b + C\right]^2} \qquad (1)$$

$$\text{(After The Peak)} \quad i = \frac{a\left[(1-b)\left(\frac{t_a}{1-r}\right)^b + C\right]}{\left[\left(\frac{t_a}{1-r}\right)^b + C\right]^2} \qquad (2)$$

Where t_b and t_a are the time measured from the peak to the left and to the right respectively, and a, b and c are constants found in the rate-duration-frequency curve formula such as

$$i_{ave} = \frac{a}{t_d{}^b + C}$$

Fig 2 - DRAINAGE PLAN OF A TYPICAL TWO-BLOCK UNIT AREA, WITH TYPE 5 LAND USE

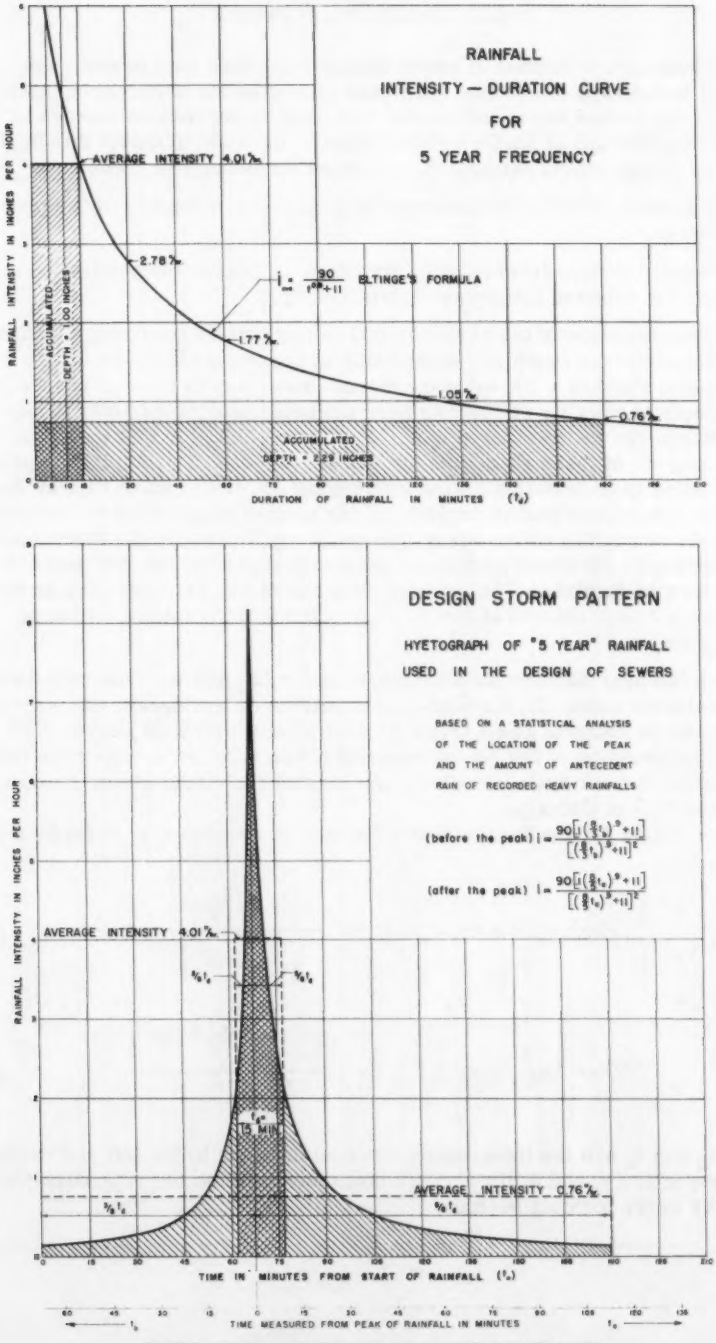


FIGURE 3 - INTENSITY DURATION CURVE AND CONCOMITANT STORM PATTERN

The value of r is a measure of the advanceness of the storm pattern. (the elapsed time, in minutes from the beginning of the design storm to its peak, divided by total duration of storm).

For this example the constants are as follows:

$$a = 90,\ b = 0.9,\ c = 11 \text{ and } r = 3/8 \tag{3}$$

The graph of the Design Storm Pattern and its interrelation with the concomitant intensity-duration curve is shown in Figure (3).

Step 3 - Abstractions From Rainfall

Infiltration

The selection of infiltration capacity curves require an examination of the type of soil and surface conditions. A study needs to be made from local or regional rainfall statistics, as to the probability of heavy rainfall on frozen ground. In Chicago, the frequency of such occurrences is much less than any frequency which can be reasonably used for design of the sewer systems. Such an occurrence is therefore, not applicable to design of Chicago's sewers.

The accepted formula for the capacity of infiltration was first suggested by Robert E. Horton,(2) and may be expressed by

$$f = f_c + (f_o - f_c)e^{-kt_f} \tag{4}$$

where f is the capacity of infiltration in inches per hour, t_f is the time in minutes measured from the beginning of the infiltration capacity curve, f_c is the constant rate to which f approaches asymptotically as time continues, f_o is the rate when t_f equals zero, and k is a positive constant.

For this example, the infiltration capacity curve for normal antecedent conditions of turf area recommended by Stifel W. Jens,(3) shall be evaluate the constants in Equation (4). Thus, Equation (4) becomes

$$f = 0.53 + 2.47\, e^{-0.07\, t_f} \tag{5}$$

and its graph is plotted in Figure 4. The area under this curve represents the accumulated mass infiltration, which can be expressed

$$F = \int_0^{t_f} \frac{f}{60}\, dt_f = 0.00883\, t_f + 0.59\left(1 - e^{-0.07 t_f}\right) \tag{6}$$

where F is the mass infiltrated in inches up to the time t_f. The graph of Equation (6) is also shown in Figure 4.

Where the rainfall at the beginning is less than the infiltration capacity curve, the actual amount absorbed by the soil is, of course, limited by the mass rainfall.

Assuming, as in Reference (4) that "overland flow and depression storage

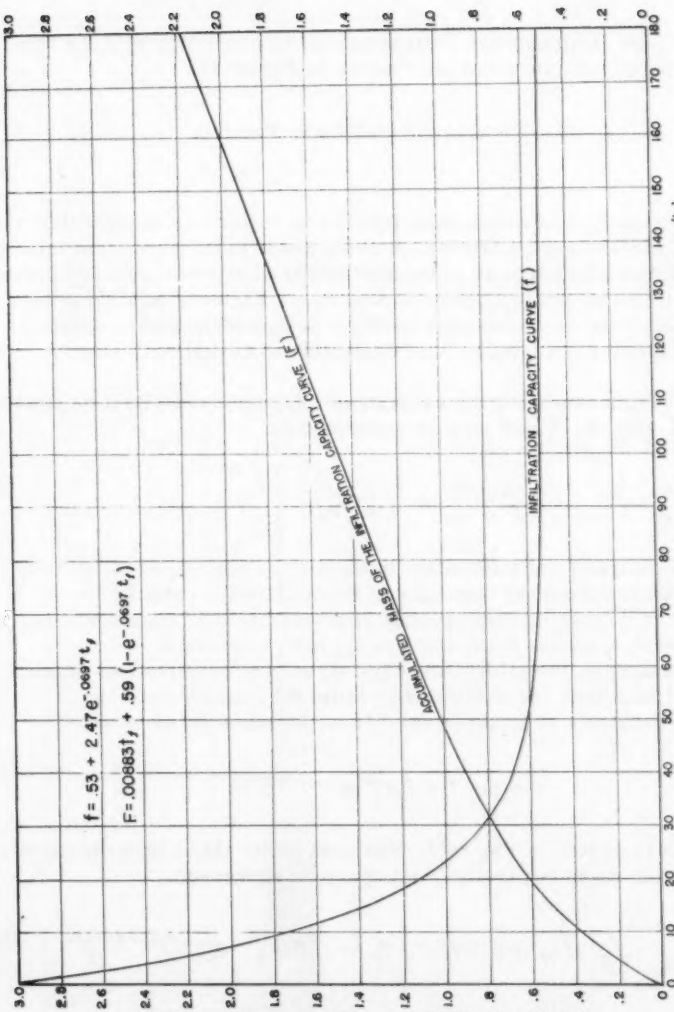

f = .53 + 2.47e^-.0697 t_f
F = .00883 t_f + .59 (1-e^-.0697 t_f)
ACCUMULATED MASS OF THE INFILTRATION CAPACITY CURVE (F)
INFILTRATION CAPACITY CURVE (f)
CAPACITY IN INCHES PER HOUR — MASS IN INCHES
TIME IN MINUTES FROM BEGINNING OF INFILTRATION CAPACITY CURVE (t_f)

Fig.4-INFILTRATION CAPACITY AND MASS CURVES FOR NORMAL ANTECEDENT CONDITIONS OF TURF AREAS

supply" commences when the antecedent mass of rainfall equals the antecedent mass of infiltration, the time of beginning of excess is found as in Figure 5a, by shifting the infiltration mass curve on its time scale until it is tangent to the rainfall mass curve. The amount of shift to bring the two curves tangent shall be termed the offset. The offset in Figure 5a was found to be 48.5 minutes by scaling. At the point of tangency where t_o equals approximately 60 minutes, the masses are the same since the ordinates are of equal heights and the rates are also the same since the slopes are equal. Figure 5b shows the rate curves of the design storm pattern, infiltration capacity, and the actual infiltration. The actual infiltration curve follows the design rainfall curve until it intercepts the offsetted infiltration capacity curve, then proceeds along the latter. It will be noted, starting at approximately 60 minutes after the beginning of the rainfall, the rainfall exceeds the infiltration, thus producing excess water which will begin to fill small depressions on the ground surface. Subtracting the ordinates of the actual infiltration curve from the rainfall rate and plotting against the same time scale, a new curve will be obtained which shall be called the "Overland Flow and Depression Storage Supply." This curve is plotted in Figure 6, along with its accumulated mass which may be obtained by subtracting the actual mass infiltrated from the accumulated mass of the design storm pattern, in Figure 5a.

Surface Depression Storage

Even with carefully graded lawns there are still many small depressions which must be filled before surface runoff begins. Where blocks contain vacant lots, some or all of the lots may be depressed below the sidewalks, thus creating large pools of depression storage. For storm sewer design however it will generally be assumed that these larger areas will be filled and graded at some later date. To evaluate the effect of depression storage an estimate must be made of the mass or average depth of rainfall which is retained in surface depressions where its dissipation can only be accomplished either by evaporation or post-storm infiltration. An assumption must also be made of the relationship between the mean depths of depressions and the proportionate areas covered by such depressions.

Since the mean depths of each depression in a drainage area is different from that of all other depressions, the runoff supply will be affected by the manner of variations of the depth of the individual depressions from the average depth of all depressions. The shallowest depressions will overflow immediately, and the deeper ones will follow in the order of their depth.

Some investigators have assumed that all depressions must be filled before overland flow supply begins, see Figure 7. This relationship would occur only if the larger depressions were grouped near the downstream end of all overland flow strips. The smaller depressions would fill and overflow into the larger ones, finally filling all depressions before overflowing into the gutter.

Other investigators,(5) have used an exponential relationship such as

$$V_d = S_d \left[1 - e^{-K_d(P-F)} \right] \tag{7}$$

where V_d equals the volume of water in surface depressions, S_d is the

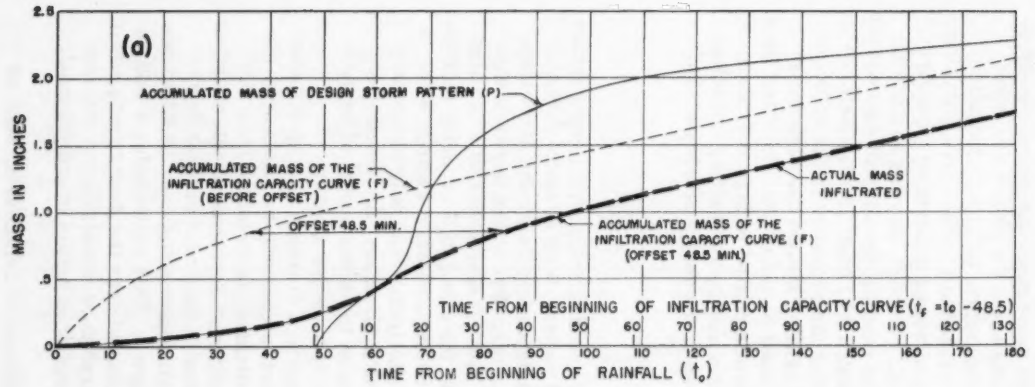

(a)
ACCUMULATED MASS OF DESIGN STORM PATTERN (P)
ACCUMULATED MASS OF THE INFILTRATION CAPACITY CURVE (F) (BEFORE OFFSET)
OFFSET 48.5 MIN.
ACTUAL MASS INFILTRATED
ACCUMULATED MASS OF THE INFILTRATION CAPACITY CURVE (F) (OFFSET 48.5 MIN.)
TIME FROM BEGINNING OF INFILTRATION CAPACITY CURVE ($t_f = t_o - 48.5$)
MASS IN INCHES
TIME FROM BEGINNING OF RAINFALL (t_o)
(b)
DESIGN STORM PATTERN (i)
INFILTRATION CAPACITY CURVE (f)
OFFSET 48.5 MIN.
EQUAL AREA
ACTUAL INFILTRATION RATE
RATE IN INCHES PER HOUR
TIME FROM BEGINNING OF RAINFALL (t_o)

Fig. 5 — DETERMINATION OF INFILTRATION OFFSET

Fig. 5 — DETERMINATION OF INFILTRATION OFFSET

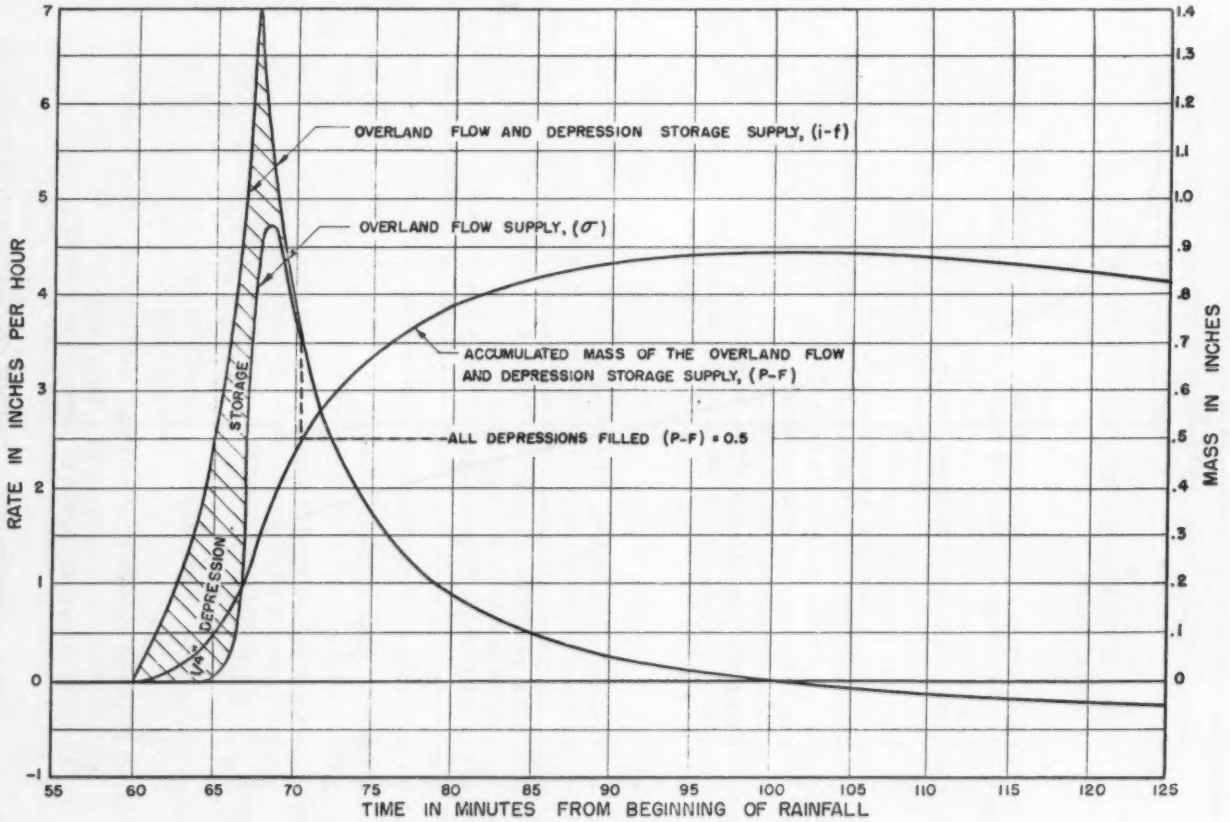


Fig. 6 — DERIVATION OF OVERLAND FLOW SUPPLY CURVE FOR PERVIOUS AREAS

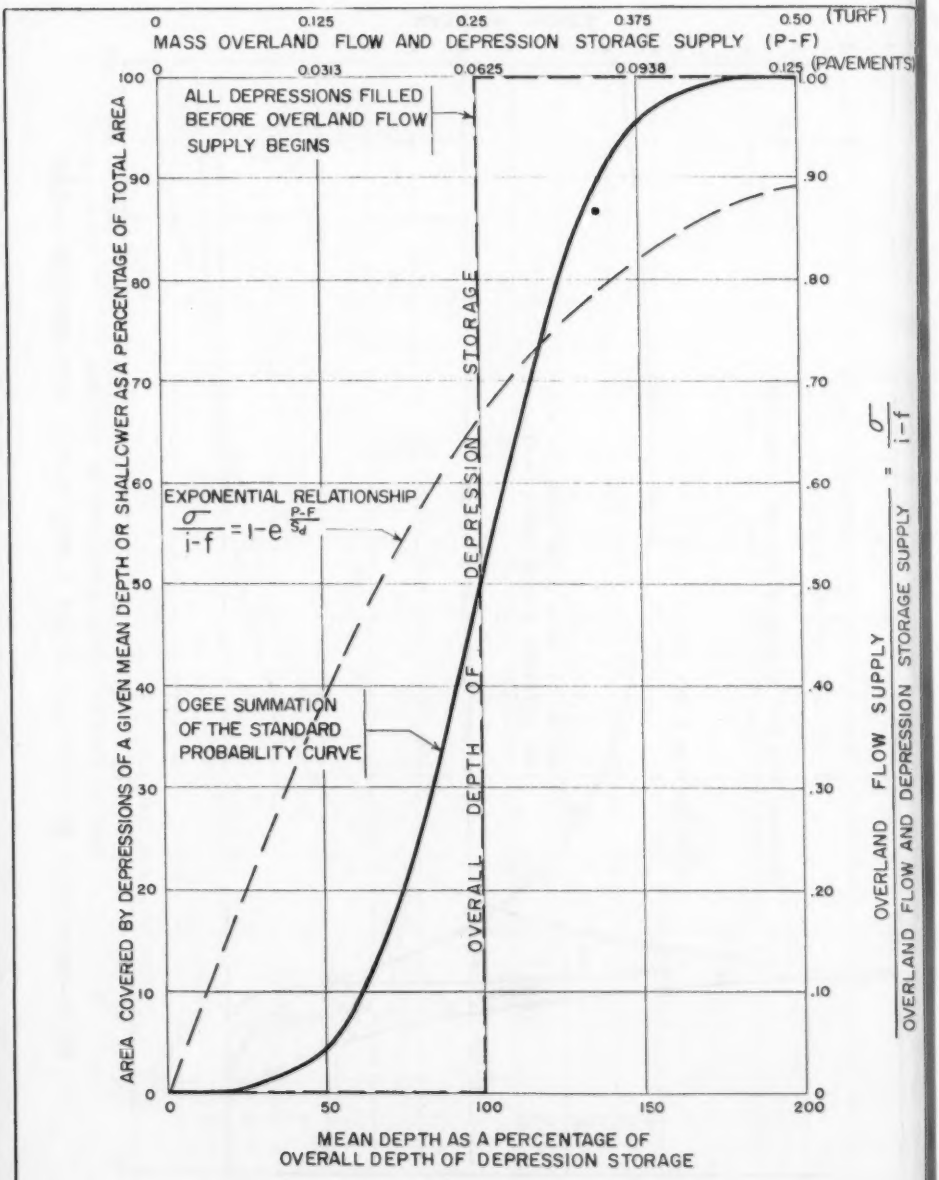

MASS OVERLAND FLOW AND DEPRESSION STORAGE SUPPLY (P-F)
0 0.125 0.25 0.375 0.50 (TURF)
0 0.0313 0.0625 0.0938 0.125 (PAVEMENTS)
ALL DEPRESSIONS FILLED BEFORE OVERLAND FLOW SUPPLY BEGINS
EXPONENTIAL RELATIONSHIP
$\frac{\sigma}{i-f} = 1 - e^{\frac{P-F}{S_d}}$
OGEE SUMMATION OF THE STANDARD PROBABILITY CURVE
OVERALL DEPTH OF DEPRESSION STORAGE
AREA COVERED BY DEPRESSIONS OF A GIVEN MEAN DEPTH OR SHALLOWER AS A PERCENTAGE OF TOTAL AREA
0 10 20 30 40 50 60 70 80 90 100
0 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00
0 50 100 150 200
MEAN DEPTH AS A PERCENTAGE OF OVERALL DEPTH OF DEPRESSION STORAGE
$\frac{\sigma}{i-f} = \frac{\text{OVERLAND FLOW SUPPLY}}{\text{OVERLAND FLOW AND DEPRESSION STORAGE SUPPLY}}$

Fig. 7 – DEPTH-DISTRIBUTION CURVE OF DEPRESSION STORAGE

overall volume of depression storage available, P-F is the mass overland flow and depression storage supply, and K_d is a storage constant. This equation can be transferred into the following rate relationship.

$$\frac{\sigma}{i-f} = \frac{\text{Overland flow supply}}{\text{Overland flow and depression storage supply}} = 1 - e^{-\frac{P-F}{S_d}} \quad (8)$$

and is plotted for comparison in Figure 7.

The authors believe, however, that some curve lying between the above two relationships would better represent the probable depth-distribution relationship for fully developed urban areas in which depressions of a given mean depth may be found at any random location. A not-too-rigorous analysis of the problem has indicated the curve may be of ogee shape. The ogee summation of the standard probability curve was thereupon adopted for the depth-distribution curve of depression storage, as shown in Figure 7.

For the following computations of overland flow supply the overall volume of depression storage is assumed equal to one quarter of an inch or pervious areas, and individual depressions range in depths from zero to one-half inch. The overall depression storage on pavement areas is set at one-sixteenth of an inch, with the depth of depressions ranging from zero to one-eighth of an inch. These assumptions seem reasonable, from observations made during heavy rainfalls.

Overland Flow Supply

The procedure has been outlined above for evaluating the deductions of infiltration and depression storage from the rainfall hyetograph to obtain the "Overland Flow Supply" curves. It is convenient now to list these deductions in a tabular form and derive, minute by minute, the numerical rates of overland flow supply. Table I shows the computation for the "Overland Flow Supply" curve. The following gives the explanation, the operation, and the symbols used in each column:

Col. (1) t_o - The time measured in minutes from the beginning of rainfall.

Col. (2) i - The rate of rainfall in inches per hour taken from the "Design Storm Pattern."

Col. (3) t_f - The time measured in minutes from the beginning of the infiltration capacity curve. Since in the present case this curve is offset by 48.5 minutes, therefore t_f equals t_o minus 48.5, see Figure 5.

Col. (4) f - The actual infiltration rate in inches per hour as previously explained, see Figure 5b

Col. (5) i-f - The overland flow and depression storage supply in inches per hour obtained by subtracting Col. (4) from Col. (2). It is unnecessary to start this table until i-f is greater than zero, since there will be no rainfall-excess to supply overland flow.

Col. (6) P - The accumulated mass of the design storm pattern in inches, see Figure 5a.

TABLE I—COMPUTATION SHEET OF OVERLAND FLOW SUPPLY FOR PERVIOUS AREAS

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
t_o	i	t_f	f	i−f	P	F	P−F	$\frac{\sigma}{i-f}$	σ
MINUTES	"/hr	MINUTES	"/hr	"/hr	INCHES	INCHES	INCHES	RATIO	"/hr
60.0	1.72	11.5	1.64	.08	.426	.426	0		0
60.5	1.81	12.0	1.60	.21	.439	.439	—	—	—
61.0	1.92	12.5	1.56	.36	.452	.452	—	—	—
61.5	2.04	13.0	1.53	.51	.466	.464	.002	—	—
62.0	2.18	13.5	1.49	.69	.482	.478	.004	—	—
62.5	2.36	14.0	1.46	.90	.500	.490	.010	—	—
63.0	2.58	14.5	1.43	1.15	.520	.502	.018	—	—
63.5	2.82	15.0	1.40	1.42	.540	.515	.025	—	—
64.0	3.11	15.5	1.37	1.74	.565	.526	.039	—	—
64.5	3.43	16.0	1.34	2.09	.594	.538	.056	—	—
65.0	3.81	16.5	1.31	2.50	.630	.550	.080	.012	.03
65.5	4.27	17.0	1.28	2.99	.66	.56	.10	.023	.07
66.0	4.87	17.5	1.25	3.62	.70	.57	.13	.053	.19
66.5	5.60	18.0	1.23	4.37	.75	.58	.17	.121	.53
67.0	6.62	18.5	1.21	5.41	.80	.59	.21	.316	1.71
67.5	8.18	19.0	1.18	7.00	.86	.60	.26	.526	3.68
68.0	7.17	19.5	1.16	6.01	.92	.61	.31	.777	4.67
68.5	6.47	20.0	1.14	5.33	.97	.62	.35	.887	4.72
69.0	5.83	20.5	1.12	4.71	1.02	.63	.39	.959	4.51
69.5	5.33	21.0	1.10	4.23	1.07	.64	.43	.99	4.19
70.0	4.87	21.5	1.08	3.79	1.11	.65	.46	.99	3.78
70.5	4.48	22.0	1.06	3.42	1.16	.66	* .50	1.00	3.42
71.0	4.14	22.5	1.05	3.09				"	3.09
71.5	3.86	23.0	1.03	2.83				"	2.83
72	3.65	23.5	1.01	2.64				"	2.64
73	3.21	24.5	.98	2.23				"	2.23
74	2.85	25.5	.95	1.90				"	1.90
75	2.58	26.5	.92	1.66				"	1.66
76	2.34	27.5	.89	1.45				"	1.45
77	2.15	28.5	.87	1.28				"	1.28
78	1.97	29.5	.84	1.13				"	1.13
79	1.82	30.5	.82	1.00				"	1.00
81	1.57	32.5	.79	.78				1.00	.78

* All Depressions Filled and Overflowing

TABLE I- continued

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
t_o	i	t_f	f	i−f	P	F	P−F	$\frac{\sigma}{1-f}$	σ
MINUTES	"/hr	MINUTES	"/hr	"/hr	INCHES	INCHES	INCHES	RATIO	"/hr
83	1.38	34.5	.76	.62				1.00	.62
85	1.21	36.5	.73	.48				"	.48
87	1.08	38.5	.70	.38				"	.38
89	.98	40.5	.67	.31				"	.31
91	.89	42.5	.65	.24				"	.24
93	.81	44.5	.64	.17				"	.17
95	.75	46.5	.63	.12				"	.12
97	.69	48.5	.62	.07				"	.07
99	.64	50.5	.60	.04				"	.04
101	.59	52.5	.59	0*				"	0*
103	.55	54.5	.59	−.04				"	−.04
105	.52	56.5	.58	−.06				"	−.06
107	.49	58.5	.57	−.08				"	−.08
109	.46	60.5	.57	−.11				"	−.11
111	.44	62.5	.56	−.12				"	−.12
113	.42	64.5	.56	−.14				"	−.14
115	.40	66.5	.56	−.16				"	−.16
117	.38	68.5	.55	−.17				"	−.17
119	.36	70.5	.55	−.19				"	−.19
121	.34	72.5	.55	−.21				"	−.21
123	.32	74.5	.54	−.22				"	−.22
125	.31	76.5	.54	−.23				"	−.23
127	.30	78.5	.54	−.24				"	−.24
129	.29	80.5	.54	−.25				"	−.25
131	.28	82.5	.54	−.26				"	−.26
133	.27	84.5	.54	−.27				"	−.27
135	.26	86.5	.54	−.28				1.00	−.28

*Hereafter the infiltration rate exceeds the rainfall rate and the negative signs indicate a withdrawal of exisiting water formerly accumulated on the surface and in depressions.

Col. (7) F - The accumulated mass of actual infiltration in inches, see Figure 5a.

Col. (8) P-F- The accumulated mass of the overland flow and depression storage supply in inches, see Figure 6.

Col. (9) $\frac{\sigma}{i-f}$ - The ratio of the "Overland Flow Supply" to the "Overland Flow and Depression Storage Supply." This value can be obtained from the ogee summation of the standard probability curve, Figure 7, by applying the accumulated mass of P-F, from Col. (8) using the horizontal ordinates for "Turf" and reading $\frac{\sigma}{i-f}$.

Col. (1) σ - The overland flow supply rate in inches per hour, obtained by multiplying Col. (5) by Col. (9).

It will be noted that the table begins at 60 minutes --- well before the peak rainfall rate --- and that the choice of time interval in Table I was one-half minute up to 72 minutes, one minute between 72 and 79 minutes, and two minutes after 79 minutes. This was done to define the curve more closely at the rapidly varying peak. Values from Col. (5), "Overland-Flow-and-Depression Storage Supply," and from Col. (10), "Overland Flow Supply," are shown graphically in Figure 6.

Step 4 - Overland Flow Along Elemental Strips

The "Overland Flow Supply" or "Rainfall-excess" curves have been developed by the deduction of retention and infiltration from the "Design Storm Pattern." As this supply increases, it builds up a sheet of water over the surface, and the water begins to flow towards the collecting gutter. This movement of the water is called "Overland Flow" and the volume of transient water on the surface is termed "Surface Detention."

Mr. Carl F. Izzard,[6] found, in experimental work done by the Public Roads Administration and the Soil Conservation Service, that under steady flow conditions, the average depth of flow is proportional to the cubic root of the discharge at the end of the strip, and inversely proportional to the cubic root of the slope of the surface.

Thus,
$$\frac{D_e}{\ell} \propto \frac{q_e^{1/3}}{S_o^{1/3}}$$

where D_e is the detention on the surface in cubic feet per foot of width, ℓ is the length of strip in feet; q_e is the "Overland Flow Runoff" at equilibrium or under steady flow conditions, in cubic feet per second, and S_o is the slope of the ground surface. The general equation for detention then is

$$D_e = K \ell q_e^{1/3} \qquad (9)$$

where K contains the slope and a roughness factor.
He also found that the roughness factor was not a constant but that it varied with the intensity of the rainfall.

$$K = \frac{(0.0007\,i + C)}{S_0^{1/3}} \tag{10}$$

Where i is the intensity of rainfall and C is the roughness factor when i is equal to zero. Equation (9) may then be written as

$$D_e = \frac{0.0007\,i + C}{S_0^{1/3}}\; \ell\, q_e^{1/3} \tag{11}$$

The values of C for surfaces of different roughness were determined experimentally by Izzard and others,[6] as follows:

Very smooth asphalt pavement	0.007
Tar and sand roofing paper	0.0075
Crushed slate roofing paper	0.0082
Concrete pavement	0.012
Tar and gravel pavement	0.017
Closely clipped sod	0.046
Dense blue grass turf	0.060

Routing Procedure

Equation (11) relates the detention or storage on the surface as a function of the rate of runoff and rainfall intensity with C, S_0 and ℓ constant for a given strip. It is assumed that the relationship also holds fairly close for unsteady flow conditions. To the writers' knowledge, no exact solution of the varied flow equation has been achieved either analytically or experimentally, but probably the difference in detention of varied flow and steady flow will not be of large magnitude. Then for unsteady flow, dropping the subscript "e," in Equation (11), and changing the units of D from cubic feet per foot of width, to inches depth per unit area, and the units of q from cubic feet per second per foot of width to inches per hour per unit area.
Equation (11) becomes

$$D = 0.342\left(\frac{0.0007\,i + C}{S_0^{1/3}}\right) \ell^{1/3} q^{1/3} \tag{12}$$

To determine the "Overland Flow Runoff" hydrograph from the "Overland Flow Supply" curve, a storage routing procedure is used similar to that used to route floods in river hydrology. The equation of continuity or storage equation takes the following form:

$$\sigma - q = 60\;\frac{dD}{dt_o} \tag{13}$$

where σ and q are the inflow and outflow respectively, in inches per hour, D the surface detention in inches, and t_o the time measured from the beginning of the rainfall in minutes. Writing this equation in an incremental form

$$\frac{1}{2}\left(\sigma_1 + \sigma_2\right) - \frac{1}{2}\left(q_1 + q_2\right) = 60\ \frac{D_2 - D_1}{\Delta t_o} \tag{14}$$

where the subscripts "1" and "2" indicate the value at the beginning and end of the routing period Δt_o. Transposing all of the known values to the left-hand side of the equation,

$$\frac{1}{2}\left(\sigma_1 + \sigma_2\right) - \frac{q_1}{2} + 60\,\frac{D_1}{\Delta t_o} = \frac{q_2}{2} + 60\,\frac{D_2}{\Delta t_o}$$

or

$$\sigma_{ave} - q_1 + \left(\frac{q_1}{2} + 60\,\frac{D_1}{\Delta t_o}\right) = \left(\frac{q_2}{2} + 60\,\frac{D_2}{\Delta t_o}\right)$$

and designating

$$F\left(q, i, \Delta t_o\right) = \left(\frac{q}{2} + 60\,\frac{D}{\Delta t_o}\right) \tag{15}$$

where F is a function of q with i and Δt_o as parameters, and the physical properties of the overland flow strip. This function can be graphically represented as shown in the example in Figure 8, for the elemental strips of overland flow. Then the routing equation becomes

$$\sigma_{ave} - q_1 + F_1 = F_2 \tag{16}$$

Since all the terms on the left-hand side of Equation (16) are known, the value of F_2 is found. Entering this value into the chart, Figure 8, with the appropriate parametric curve and constants q_2 can be determined. Then q_2 and F_2 for this period becomes q_1 and F_1 for the subsequent period and the above process is repeated. Tabulation forms have been set up to carry on this computation procedure as shown in Table II. As an example of computation, consider the increment of time from 70 minutes to 70-1/2 minutes. The average supply in this period is 3.58" per hour, and the q_1 and F_1 previously computed are 0.244 and 25.945 respectively. Now by, the routing equation, Equation (16),

$$F_2 = 3.58 - 0.244 + 25.945 = 29.281$$

Entering this number into the chart, Figure 8, the parametric curve, for i = 4.47"/hr and Δt_o = 1/2 minute gives q_2 = 0.353"/hr. This procedure may then be repeated for the next time increment, and so on.

In Table I the values of "Overland Flow Supply" were computed at time intervals so chosen as to define the rapidly varying peak and to eliminate unnecessary computations in the flatter portions. These same intervals have been carried on to overland flow as the routing periods. It will be noted in the computation sheet, Table II, that each time the increment Δt_o is changed,

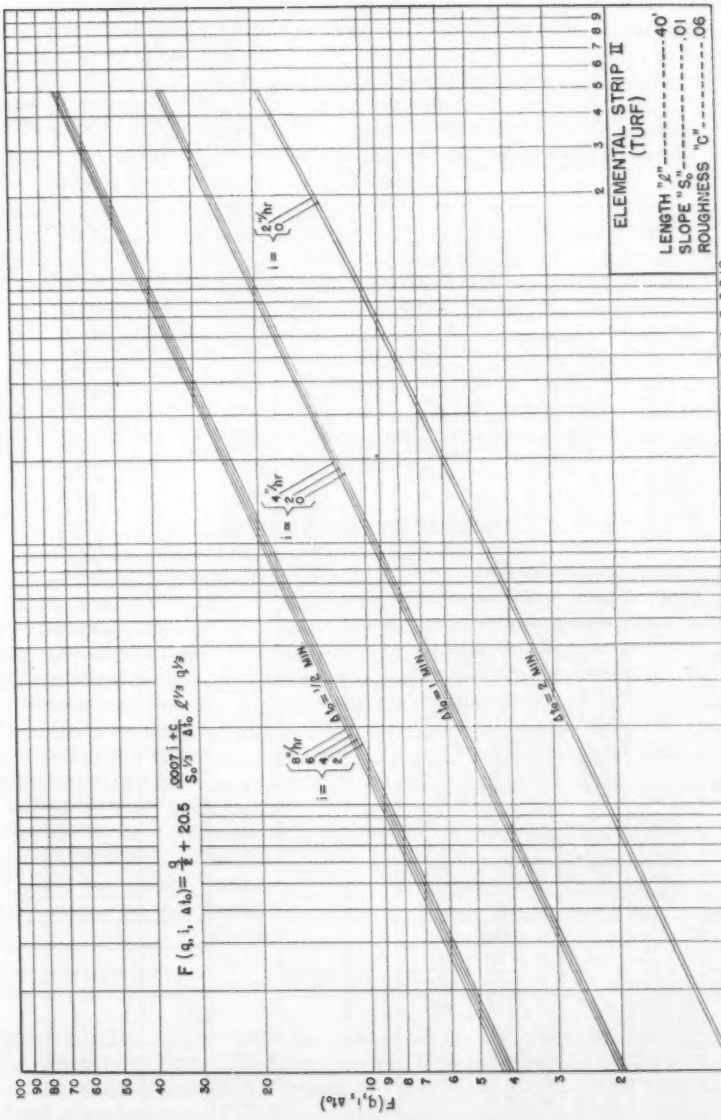

ELEMENTAL STRIP II
(TURF)
LENGTH "ℓ"----------40'
SLOPE "S₀"----------.01
ROUGHNESS "C"----------.06
F (q, i, Δt₀) = q/2 + 20.5 .0007 i + c / S₀^1/3 Δt₀ ℓ^1/3 q^1/3
i = 8"/hr 6 4 2
i = 4"/hr 2 0
i = 2"/hr 0
Δt₀ = 1/2 MIN
Δt₀ = 1 MIN
Δt₀ = 2 MIN
F(q, i, Δt₀)
q "/hr

Fig. 8 – ROUTING CHART FOR ELEMENTAL STRIP II

TABLE II COMPUTATION SHEET FOR ROUTING OVERLAND FLOWS THROUGH ELEMENTAL STRIP II

ROUTING EQUATION
$\sigma_{ave.} - q_1 + F_1 = F_2$

(TURF)
LENGTH "L" ——— 40'
SLOPE "S_o" ——— .01
ROUGHNESS "C" ——— .06

Δt_o	TIME t_o min.	AVE. SUPPLY $\sigma_{ave.}$ "/hr.	RUNOFF q "/hr.	F(q, i, Δt_o) "/hr.	RAINFALL INTENSITY i · "/hr.
Δt_o = ½ MIN.	64	.000	—	—	3.09
	64½	.015	—	—	3.42
	65	.05	—	.015	3.70
	65½	.13	—	.065	4.25
	66	.36	—	.195	4.82
	66½	1.12	—	.555	5.58
	67	2.69	—	1.675	6.57
	67½	4.17	.001	4.365	8.18
	68	4.69	.008	8.534	7.18
	68½	4.62	.032	13.216	6.44
	69	4.39	.070	17.804	5.32
	69½	3.97	.149	22.124	5.30
	70	3.58	.244	25.945	4.86
	70½	3.25	.353	29.281	4.47
	71	2.96	.476	32.178	4.14
	71½	2.71	.596	34.662	3.88
	72		.715	36.776	3.62
Δt_o = 1 MIN.	72	2.41	.715	18.551	3.62
	73	2.08	.925	20.246	3.21
	74	1.79	1.10	21.401	2.87
	75	1.56	1.22	22.091	2.58
	76	1.36	1.28	22.431	2.35
	77	1.20	1.30	22.511	2.14
	78	1.06	1.29	22.411	1.97
	79	.	1.27	22.181	1.82
Δt_o = 2 MIN.	79	.890	1.27	11.408	1.82
	81	.700	1.16	11.028	1.58
	83	.550	1.03	10.568	1.37
	85	.435	.92	10.088	1.21
	87	.340	.805	9.603	1.08
	89	.263	.705	9.138	.97
	91	.200	.62	8.696	.88
	93	.145	.543	8.276	.81
	95	.095	.472	7.878	.74
	97	.050	.413	7.501	.68
	99	.017	.36	7.138	.64
	101	-.016	.31	6.795	.60
	103	-.045	.267	6.469	.56
	105	-.071	.234	6.157	.52
	107	-.093	.202	5.852	.49
	109	-.113	.172	5.557	.46
	111		.150	5.272	.43

Δt_o	TIME t_o min.	AVE. SUPPLY $\sigma_{ave.}$ "/hr.	RUNOFF q "/hr.	F(q, i, Δt_o) "/hr.	RAINFALL INTENSITY i "/hr.
Δt_o = 2 MIN.	111		—	4.990	.43
	113	-.132	.127	4.713	.41
	115	-.150	.108	4.439	.39
	117	-.166	.090	4.167	.37
	119	-.182	.075	3.896	.36
	121	-.196	.062	3.625	.34
	123	-.209	.050	3.354	.33
	125	-.221	.039	3.082	.32
	127	-.233	.030	2.808	.31
	129	-.244	.023	2.530	.30
	131	-.255	.017	2.249	.29
	133	-.264	.012	1.965	.28
	135	-.272	.008	1.676	.27
	137	-.281	.005	1.383	.26
	139	-.288	.003	1.084	.25
	141	-.296	.001	.780	
	143	-.303		.471	
		-.309			

two spaces are alloted for the time of the change. Since F is a function of q, i and Δt_o, its value reflects the change in any of its parameters, but for a given instance the values of q and i are unique, so F reflects only the change in Δt_o. Thus, when a change of time increment occurs, the new F value can be obtained from that set of curves corresponding to the new time interval in Figure 8, while holding q and i unchanged. This operation must be done at every change of time interval.

The overland flow supply and runoff hydrographs for the various elemental strips are shown in Figure 9. These were plotted from computations similar to that in Table II. In this graphical form it is easy to visualize the relative reduction and delaying effects on the peak of the supply hydrographs when routed through the overland flow storage.

The computation procedure for impervious surfaces is the same as that outlined for the pervious surfaces, with the exception that the rainfall is not reduced by surface infiltration, and therefore in Table I, columns 3, 4, 5 and 7 need not be filled in. Since usually only a small amount of depression storage is assumed on the impervious areas, the depressions are filled early in the rainfall, thereafter the rainfall rate i becomes the supply for overland flow σ. The routing through overland flow detention is performed in the same manner as that used on pervious areas. A routing curve is prepared similar to that shown in Figure 8, with of course, the use of the appropriate constant for the impervious surface. Figure 10 shows the overland flow from the pavement area, Strip Number IV.

Step 5 - Gutter Routing

The mixed flow from the pavement and lawns must now be routed through the street gutter detention to determine the hydrograph of flow entering the inlet or catch basin. Unit runoff rates from the overland flow hydrographs of the elemental strips of pavement and lawn are multiplied by the respective tributary area of each, and the products added together to give the gutter inflow hydrograph in cubic feet per second. The inflow hydrograph is assumed to be uniformly distributed along the length of the gutter. The gutter supply starts at zero at the upstream end of the gutter and increased with distance to the instantaneous rate of the inflow hydrograph at the downstream end of the gutter or inlet grating.

For the routing process, it is necessary to determine the instantaneous volume of storage in the gutter for a given inflow and outflow rate.

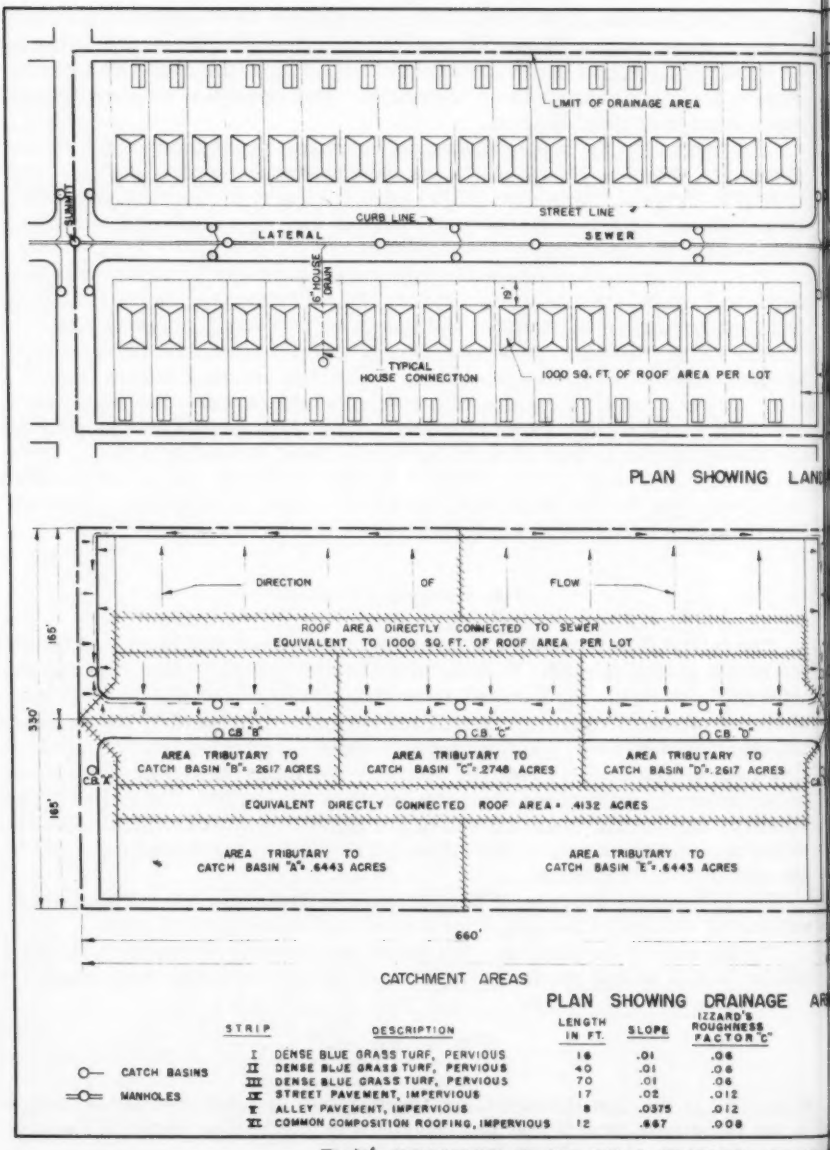
Taking any-cross-section of the gutter and splitting it into vertical elemental strips so that the flow through each such strip can be expressed

$$dQ = Vh\,dw \tag{17}$$

Where dQ is the flow through the elemental area in cubic feet per second, v is the velocity of flow in feet per second, h is the depth of water in feet and dw is the width of said strip in feet.

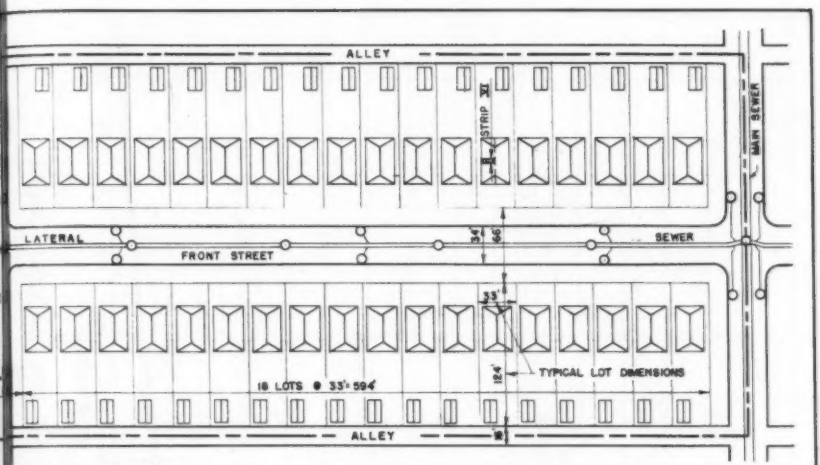
Applying Manning's Formula for velocity and assuming the hydraulic radius is equal to the depth h, the above expression becomes,

$$dQ = \frac{1.486}{n} h^{\frac{2}{3}} S_g^{\frac{1}{2}} h\,dw$$

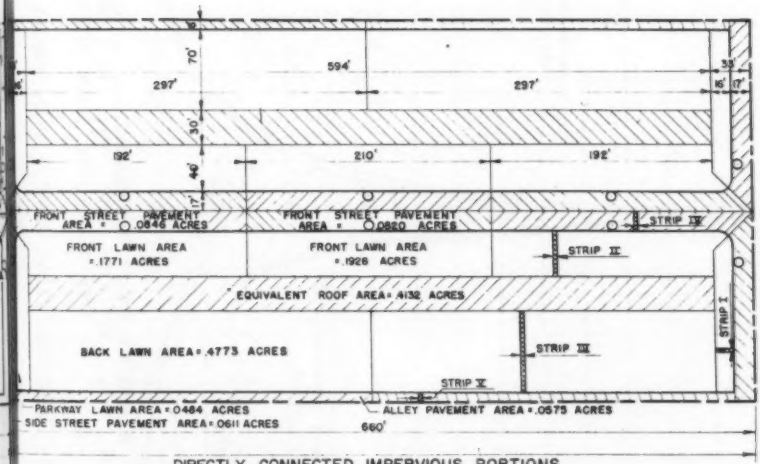

LIMIT OF DRAINAGE AREA
STREET LINE
CURB LINE
SUMMIT
LATERAL
SEWER
6" HOUSE DRAIN
TYPICAL HOUSE CONNECTION
1000 SQ. FT. OF ROOF AREA PER LOT
PLAN SHOWING LAND USE AND SE
DIRECTION OF FLOW
ROOF AREA DIRECTLY CONNECTED TO SEWER
EQUIVALENT TO 1000 SQ. FT. OF ROOF AREA PER LOT
CB "B"
CB "C"
CB "D"
AREA TRIBUTARY TO CATCH BASIN "B" = 2617 ACRES
AREA TRIBUTARY TO CATCH BASIN "C" = 2748 ACRES
AREA TRIBUTARY TO CATCH BASIN "D" = 2617 ACRES
CB "A"
EQUIVALENT DIRECTLY CONNECTED ROOF AREA = 4132 ACRES
AREA TRIBUTARY TO CATCH BASIN "A" = 6443 ACRES
AREA TRIBUTARY TO CATCH BASIN "E" = 6443 ACRES
165'
330'
165'
660'
CATCHMENT AREAS
PLAN SHOWING DRAINAGE AREAS AND DIRE
CATCH BASINS
MANHOLES

STRIP	DESCRIPTION	LENGTH IN FT.	SLOPE	IZZARD'S ROUGHNESS FACTOR "C"
I	DENSE BLUE GRASS TURF, PERVIOUS	16	.01	.06
II	DENSE BLUE GRASS TURF, PERVIOUS	40	.01	.06
III	DENSE BLUE GRASS TURF, PERVIOUS	70	.01	.06
IX	STREET PAVEMENT, IMPERVIOUS	17	.02	.012
X	ALLEY PAVEMENT, IMPERVIOUS	8	.0375	.012
XI	COMMON COMPOSITION ROOFING, IMPERVIOUS	12	.667	.008

Fig. 9 - DRAINAGE PLAN OF A TYPICAL TWO-BLOCK UN



LAND SE AND SEWERAGE



DIRECTLY CONNECTED IMPERVIOUS PORTIONS

E AR AND DIRECTION OF FLOW

TOTAL IMPERVIOUSNESS

PERVIOUS AREAS	5.37ACRES	53.7 %
IMPERVIOUS AREAS		
DIRECTLY CONNECTED	3.60ACRES	36.0 %
DISCHARGING ONTO PERVIOUS AREAS *	1.03ACRES	10.3 %
UNIT AREA	10.00ACRES	100.0 %

* INCLUDES GARAGE ROOFS, PUBLIC WALKS AND PRIVATE WALKS

TWO-BLOCK UNIT AREA, WITH TYPE 5 LAND USE

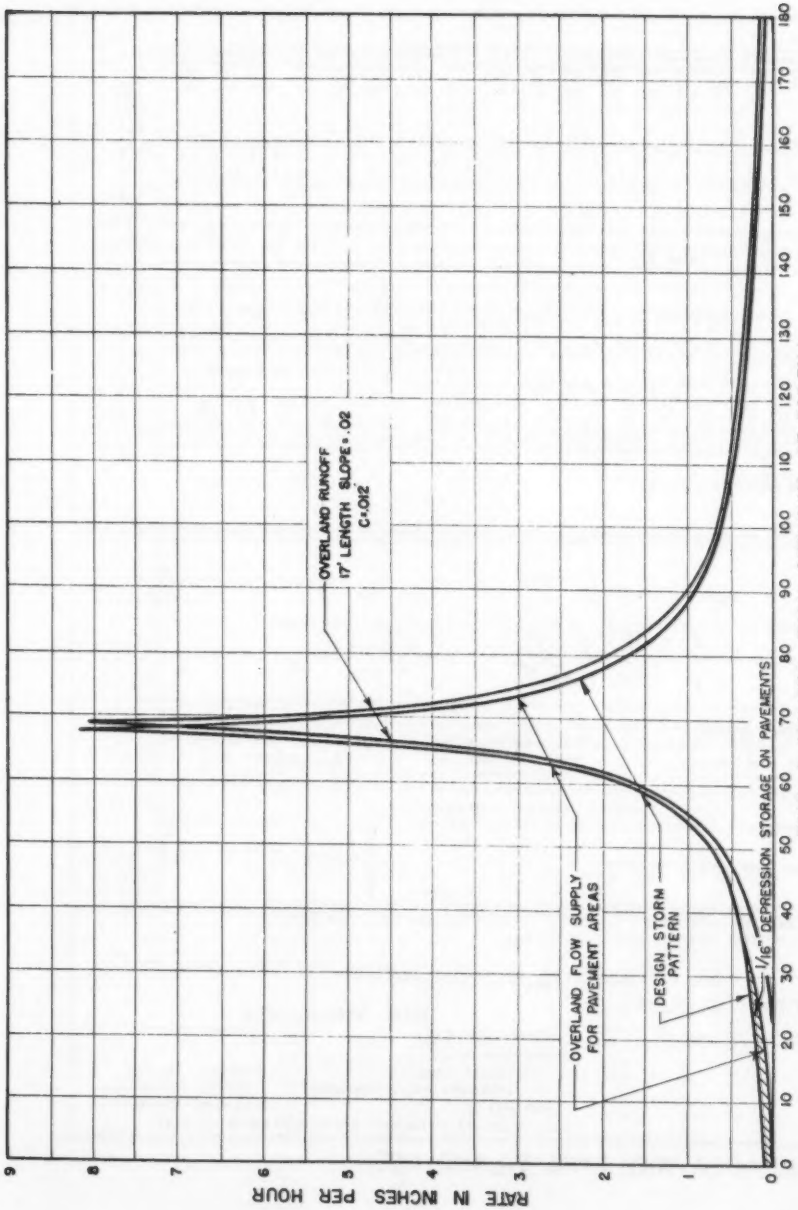

RATE IN INCHES PER HOUR
TIME IN MINUTES FROM BEGINNING OF RAINFALL
OVERLAND RUNOFF
17' LENGTH SLOPE = .02
C=.012
OVERLAND FLOW SUPPLY FOR PAVEMENT AREAS
DESIGN STORM PATTERN
1/16" DEPRESSION STORAGE ON PAVEMENTS

Fig. 10 – OVERLAND FLOW SUPPLY AND RUNOFF CURVES FOR PAVEMENT AREAS

Fig. 10 – OVERLAND FLOW SUPPLY AND RUNOFF CURVES FOR PAVEMENT AREAS

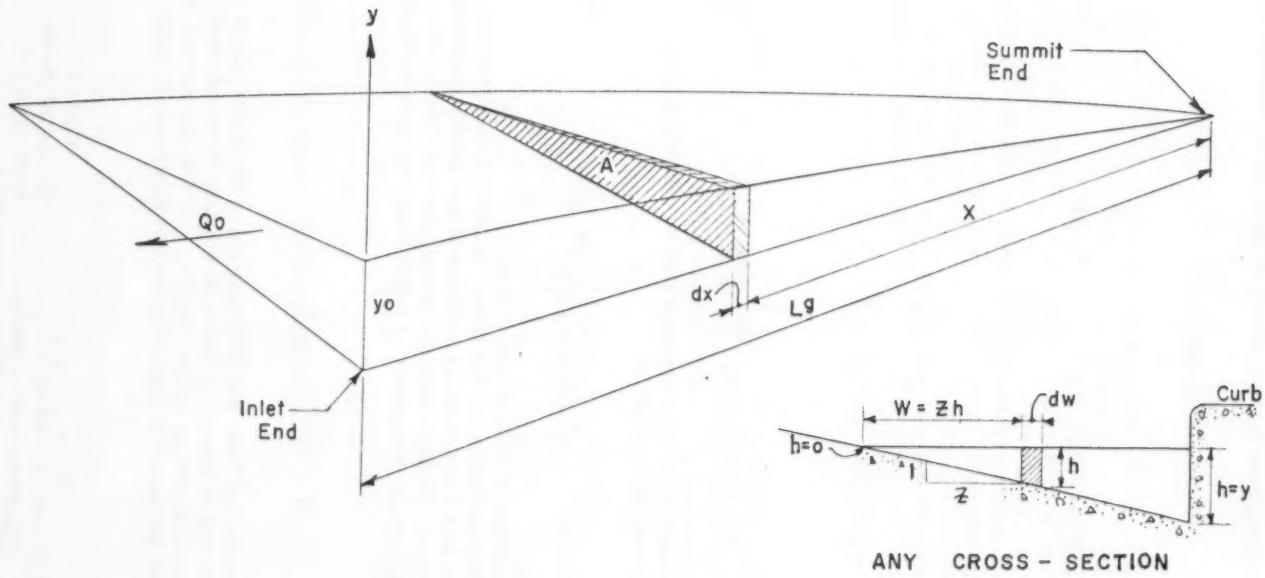


Fig. 11 – SCHEMATIC DRAWING SHOWING GUTTER CONFIGURATION

Where S_g, is approximately equal to the longitudinal bottom slope of the gutter, and n is Manning's roughness factor. Setting w equal to zh as shown in Figure 11 then,

$$dw = Z\,dh$$

Substituting

$$dQ = \frac{1.486}{n}\ S_g^{1/2}\ Z\ h^{5/3}\ dh \tag{18}$$

Integrating this expression from where the water depth is zero at the edge of its surface to y at the curb

$$Q = \int_0^y \frac{1.486}{n}\ S_g^{1/2}\ Z\ h^{5/3}\ dh$$

$$Q = \frac{0.557}{n}\ S_g^{1/2}\ Z\ y^{8/3}$$

or

$$y = \left(\frac{n}{0.557\ Z\ S_g^{1/2}}\right)^{3/8}\ Q^{3/8} \tag{19}$$

The area of any cross-section of the gutter can then be expressed as a function of the flow through that section by

$$A = \frac{Zy^2}{2} = \frac{Z}{2}\left(\frac{n}{0.557\ Z\ S_g^{1/2}}\right)^{3/4}\ Q^{3/4} \tag{20}$$

Where A is the cross-sectional area in square feet.

In order to determine the volume in the gutter, the flow through any section must be described as a function of the distance of that section from the upstream end of the gutter. First assuming a steady flow condition, where Q at any section is equal to the summation of inflow from the upstream end to that section

$$Q = Q_o\frac{X}{L_g} = I_o\frac{X}{L_g} \tag{21}$$

Where x is the distance in feet from the upstream end of the gutter, Q_o is the flow at the downstream end of the gutter, which in the case of steady flow, is equal to I_o the instantaneous rate of inflow, both in cubic feet per second, and L_g is the total length of the gutter in feet. The volume of storage in cubic feet is

$$S_e = \int_0^{L_g} A\,dX \tag{22}$$

Substituting Equation (21) into Equation (19) and replacing A

$$S_e = \int_0^{L_g} \frac{z}{2} \left(\frac{n}{0.557 z S_g^{1/2}}\right)^{3/4} Q_o^{3/4} \left(\frac{x}{L_g}\right)^{3/4} dx \qquad (23)$$

$$S_e = \frac{n^{3/4} z^{1/4} L_g}{2.26 S_g^{1/3}} Q_o^{3/4}$$

Equation (23) relates the storage S_e in the gutter to the outflow Q_o if the flow in the gutter is under equilibrium condition.

For the case of varied flow, it will be assumed that the volume of storage in the gutter is a function of both the inflow and the outflow. The flow Q at the downstream end of the gutter must be equal to Q_o, moving upstream Q must approach the rate of inflow, since I-Q equals the rate of change of the volume of storage upstream of the section. Near the upstream end of the gutter, the storage is very small and the rate of changes of storage approaches zero, thus I-Q approaches zero, as shown in Figure 12. Setting the boundary conditions as

$$(Q)_{x=L_g} = Q_o \quad \text{and} \quad \left(\frac{dQ}{dx}\right)_{x=0} = \frac{I_o}{L_g}$$

The equation $Q = \frac{x}{L_g}\left[I_o - \frac{x}{L_g}(I_o - Q_o)\right]$ would fit the above boundary conditions, but would be difficult to integrate when raised to the 3/4 power. A slight modification in the expression can both fit the given boundary conditions and make it simple to integrate.

$$Q = I_o \left(\frac{x}{L_g}\right) \left\{1 - \frac{x}{L_g}\left[1 - \left(\frac{Q_o}{I_o}\right)^{3/4}\right]\right\}^{\frac{1}{3}} \qquad (24)$$

$$Q^{3/4} = I_o^{3/4}\left(\frac{x}{L_g}\right)^{3/4} - I_o^{3/4}\left(\frac{x}{L_g}\right)^{3/4}\left[1 - \left(\frac{Q_o}{I_o}\right)^{3/4}\right]$$

Combining with Equation (20) and (22)

$$S = \frac{z}{2}\left(\frac{n}{0.557 z S_g^{1/2}}\right)^{3/4} I_o^{3/4} \int_0^{L_g} \left\{\left(\frac{x}{L_g}\right)^{3/4} - \left(\frac{x}{L_g}\right)^{3/4}\left[1 - \left(\frac{Q_o}{I_o}\right)^{3/4}\right]\right\} dx$$

Integrating and Substituting Equation (23)

$$S = S_e \frac{4\left(\frac{I}{I_o}\right)^{3/4} + 7}{11} \qquad (25)$$

$$S = \frac{n^{3/4} z^{1/4} L_g}{2.26 S_g^{1/6}}\left(\frac{4}{11} I_o^{3/4} + \frac{7}{11} Q_o^{3/4}\right)$$

Equation (25) will be used to evaluate the volume of storage as a function of inflow and outflow.

Routing through the gutter will be performed similar to that of overland flow. The equation of continuity can be expressed as

FIGURE 12 — INSTANTANEOUS DISTRIBUTION OF "Q" ALONG THE GUTTER

$$I_o - Q_o = \frac{1}{60} \frac{dS}{dt_o}$$

Writing it in incremental form, dropping the subscript in Q_O and I_O, and transposing all the known values, at the beginning of the period Δt_O, to the left hand side of the equation, giving

$$\frac{1}{2}\left(I_1 + I_2\right) - \frac{1}{2} Q_1 + \frac{S_1}{60\Delta t_o} = \frac{1}{2} Q_2 + \frac{S_2}{60\Delta t_o} \qquad (26)$$

Designating

$$F_g\left(Q, I\right) = \frac{1}{2} Q + \frac{S}{60\Delta t_o} \qquad (27)$$

Where S is described by Equation (25), and F_g, being a function of inflow and outflow, and having the parameters of the physical shape and roughness of the gutter and the time increment used in the routing. A graph of the function F_g, is shown in Figure 13 and will be used for routing the flow through the gutter storage.

By the substitution of Equation (27) into Equation (26) the routing formula becomes

$$I_{ave.} - Q_1 + F_1 = F_2 \qquad (28)$$

Table III shows an example of the gutter routing using the graph in Figure 13 and Equation (28).

Step 6 - Routing Through Lateral Sewer

Several methods of routing through the lateral sewer were examined. Storage routing procedures similar to that used to compute overland flow and gutter flow were explored and compared with the time-offset methods as described below.

Due to its simplicity, the method of routing through the lateral sewer by time-offset was adopted. Every particle of water is assumed to travel at a constant velocity along the distance from where it enters the underground sewer system, to the outlet end of the lateral sewer. Each hydrograph is shifted or offset by the time taken to traverse this distance. Referring back to Figure 2, it can be seen that there are many points of entry of flow into the lateral sewer. Since each of these inflow points have different time of travel to the outlet, the peaks of the hydrographs are offset by different amounts, causing the summation of these offset hydrographs to be less than the summation of all the inflow hydrographs. The handling of up to thirty-six pair of house drain hydrographs from the roofs and ten pair of gutter hydrographs would be laborious. A simplified time-offset method will be outlined in the following paragraphs, which is much easier to use and will produce almost the same results.

All of the inflow hydrographs from the gutter and roofs are added together forming the total inflow hydrograph to the lateral sewer system. The tabular

FIGURE 12 — INSTANTANEOUS DISTRIBUTION

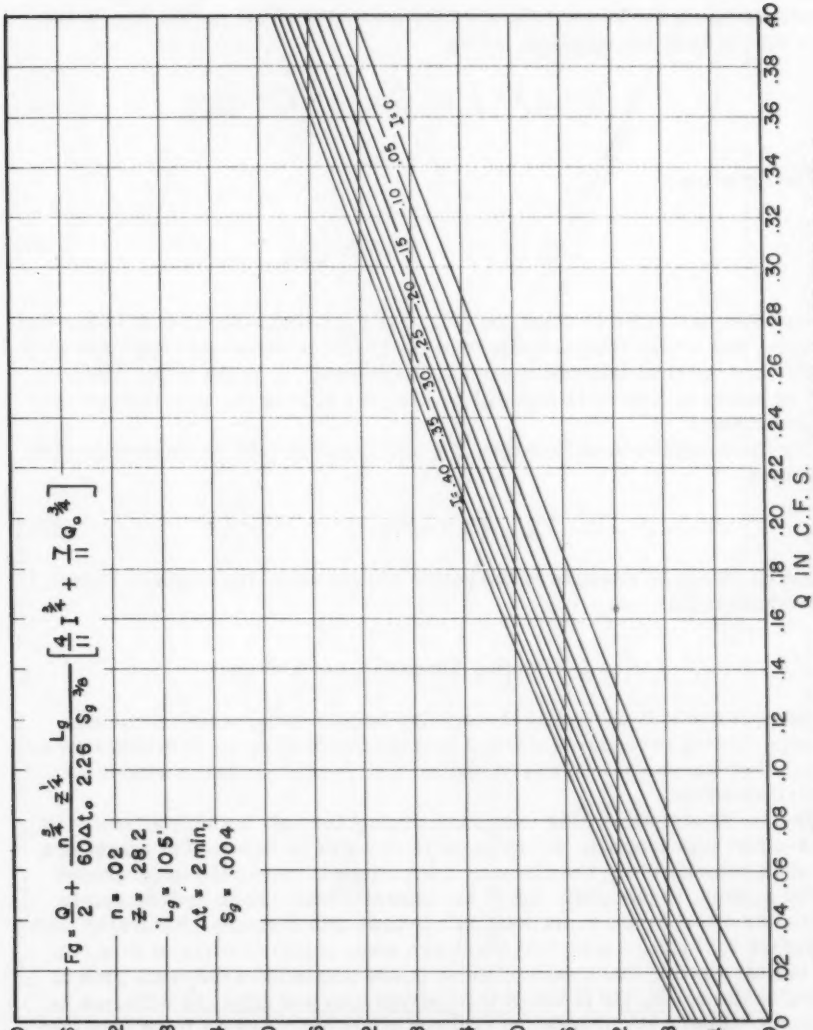


Fig. 13 – STORAGE ROUTING CURVE FOR FRONT STREET GUTTER

TABLE III – ROUTING OF GUTTER FLOW

t_o Time in Min.	I Inflow in C.F.S.	I Ave. Ave. Inflow In C.F.S.	Q Outflow In C.F.S.	F Storage In C.F.S.
.5	—	—	—	—
2.5	—	—	—	—
4.5	—	—	—	—
6.5	—	—	—	—
8.5	—	—	—	—
10.5	—	—	—	—
12.5	—	—	—	—
14.5	—	—	—	—
16.5	—	—	—	—
18.5	—	—	—	—
20.5	.0001	.0002	.0001	.0002
22.5	.0003	.0006	.0004	.0007
24.5	.0010	.0018	.0008	.0021
26.5	.0026	.0038	.0018	.0051
28.5	.005	.0063	.0042	.0096
30.5	.0077	.0089	.0076	.0143
32.5	.0101	.0110	.0102	.0177
34.5	.0119	.0126	.0121	.0202
36.5	.0134	.0141	.0136	.0222
38.5	.0147	.0154	.0150	.0240
40.5	.0161	.0169	.0165	.0259
42.5	.0177	.0186	.0181	.0281
44.5	.0195	.0206	.0200	.0305
46.5	.0216	.0229	.0223	.0335
48.5	.0243	.0259	.0251	.0370
50.5	.0275	.0295	.0287	.0413
52.5	.0315	.0341	.0332	.0467
54.5	.0367	.0401	.0391	.0537
56.5	.0436	.0483	.0471	.0629
58.5	.0530	.0598	.0584	.0757
60.5	.0667	.0771	.0755	.0944
62.5	.0876	.1052	.1037	.1242
64.5	.1229	.1576	.1568	.1781
66.5	.1922	.2672	.2729	.2885
68.5	.3422	.2899	.3155	.3055
70.5	.2377	.2285	.2036	.2185
72.5	.2193	.2166	.2216	.2315
74.5	.2138	.2094	.2074	.2193
76.5	.2049	.1979	.1979	.2098
78.5	.1908	.1824	.1813	.1943
80.5	.1739	.1652	.1643	.1782
82.5	.1565	.1482	.1473	.1621
84.5	.1399	.1323	.1317	.1471
86.5	.1247	.1180	.1176	.1334
88.5	.1112	.1633	.1051	.1211
90.5	.0993	.0940	.0940	.1101
92.5	.0888	.0842	.0843	.1003
94.5	.0796	.0756	.0758	.0915
96.5	.0715	.0679	.0682	.0837
98.5	.0644	.0612	.0616	.0769
100.5	.0581	.0553	.0557	.0704
102.5	.0525	.0500	.0504	.0647
104.5	.0475	.0453	.0457	.0596
106.5	.0431	.0411	.0415	.0549
108.5	.0391	.0373	.0378	.0507
110.5	.0355	.0339	.0344	.0469
112.5	.0324	.0309	.0314	.0434
114.5	.0295	.0282	.0287	.0402
116.5	.0269	.0257	.0262	.0373
118.5	.0246	.0235	.0240	.0346
120.5	.0225	.0216	.0220	.0322
122.5	.0206	.0198	.0202	.0300
124.5	.0189	.0182	.0186	.0280
126.5	.0174	.0168	.0171	.0261
128.5	.0161	.0155	.0158	.0245
130.5	.0149	.0144	.0147	.0230
132.5	.0138	.0134	.0137	.02168
134.5	.0129	.0125	.0128	.0205
136.5	.0121	.0117	.0120	.0195
138.5	.0114	.0111	.0113	.0186
140.5	.0108	.0105	.0107	.1776
142.5	.0102	.0100	.0102	.1708
144.5	.0098	.0096	.0010	.1649
146.5	.0094	.0093	.0009	.1598
148.5	.0091	.0090	.0009	.1555
150.5	.0088	.0087	.0009	.1516
152.5	.0086	.0084	.0009	.1481
154.5	.0083	.0082	.0008	.1447
156.5	.0081	.0080	.0008	.1415
158.5	.0079	.0078	.0008	.1384
160.5	.0078	.0076	.0008	.1355
162.5	.0076	.0074	.0007	.1327
164.5	.0073	.0072	.0007	.1300
166.5	.0071	.0070	.0007	.1274
168.5	.0069	.00685	.0007	.1250
170.5	.0068	.0067	.0007	.1226
172.5	.0066	.00655	.0007	.1203
174.5	.0065	.0064	.0006	.1181
176.5	.0063	.00625	.0006	.1160
178.5	.0062			

Fig. 15 – STORAGE ROUTING CURVE

TABLE IV—ROUTING THROUGH LATERAL SEWER

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
t_o	24x Gutter Type B,C,D	8x Gutter Type A & E	Roof Hydrograph 1"/hr x 1.65 Acres	I	Mass I at t_o	Mass I at $t_o - 7\frac{1}{3}$	Δ Mass I	$Q = \frac{\Delta \text{ Mass } I}{7\frac{1}{3}}$
Min.	c.f.s.	c.f.s.	c.f.s.	c.f.s.	cfs x Min.	cfs x Min.	cfs x Min.	c.f.s.
0.5			.2032	.2032	0.0508	—	0.0508	0.0069
2.5			.2171	.2171	.4711	—	.4711	.0642
4.5			.2251	.2251	.9133	—	.9133	.1245
6.5			.2337	.2337	1.3721	—	1.3721	.18710
8.5			.2429	.2429	1.8487	.1226	1.7261	.23539
10.5			.2529	.2529	2.3435	.5434	1.8001	.24548
12.5			.2637	.2637	2.8611	.9893	1.8718	.25525
14.5			.2754	.2754	3.4002	1.4511	1.9491	.26579
16.5			.2881	.2881	3.9637	1.9310	2.0327	.27720
18.5			.3020	.3020	4.5537	2.4300	2.1237	.28959
20.5			.3172	.3172	5.1729	2.9499	2.223	.30307
22.5			.3339	.3339	5.8240	3.4940	2.330	.31779
24.5			.3524	.3524	6.5103	4.0613	2.449	.3339
26.5			.3728	.3728	7.2355	4.6564	2.5791	.3517
28.5	.0152		.3908	.4060	8.0143	5.2863	2.728	.3720
30.5	.1006	.0090	.4147	.5243	8.9446	5.9486	2.996	.4085
32.5	.1814	.0558	.4434	.6806	10.1495	6.6435	3.506	.4781
34.5	.2434	.1193	.4758	.8385	11.6686	7.3746	4.294	.5856
36.5	.2895	.1817	.5130	.9842	13.4913	8.1643	5.327	.7265
38.5	.3213	.2252	.5455	1.0920	15.5675	9.0295	6.538	.8915
40.5	.3550	.2692	.5951	1.2193	17.8788	10.2858	7.593	1.0354
42.5	.3948	.3184	.6628	1.3760	20.4741	11.8582	8.6159	1.1749
44.5	.4342	.3559	.7312	1.5213	23.3714	13.7325	9.6389	1.3144
46.5	.4803	.3954	.8132	1.6889	26.5816	15.8816	10.700	1.4591
48.5	.5357	.4400	.9128	1.8885	30.1590	18.2910	11.868	1.6184
50.5	.6033	.4927	1.0362	2.1322	34.1797	20.4667	13.713	1.870
52.5	.6878	.5568	1.1919	2.4365	38.7484	22.7914	15.957	2.176
54.5	.7955	.6371	1.3935	2.8261	44.0110	26.4400	17.571	2.396
56.5	.9367	.7404	1.6625	3.3396	50.1767	29.8777	20.299	2.768
58.5	1.1283	.8776	2.0357	4.0416	57.5579	33.9369	23.621	3.221
60.5	1.4000	1.0673	2.5806	5.0479	66.5474	38.5924	27.955	3.812
62.5	1.8090	1.3435	3.4343	6.5868	78.2821	43.6911	34.591	4.717
64.5	2.4790	1.7761	4.9238	9.1789	94.0478	49.5198	44.528	6.072
66.5	3.7351	2.5316	8.0813	14.3480	117.575	54.795	62.780	8.561
68.5	6.6590	4.2578	9.1982	20.1150	152.038	62.835	89.203	12.164
70.5	6.0797	6.2369	6.4734	18.7900	190.943	73.669	117.274	15.992
72.5	5.3714	6.0575	4.9241	16.3530	226.086	96.623	129.463	17.654
74.5	5.1850	6.4449	3.9231	15.5530	257.992	127.305	130.687	17.821
76.5	5.0213	6.8945	3.2272	15.1430	288.688	165.591	123.097	16.786
78.5	4.7335	7.0085	2.7200	14.4620	318.293	206.857	111.436	15.196
80.5	4.3531	6.7716	2.3353	13.4600	346.215	240.696	105.519	14.389
82.5	3.9400	6.3164	2.0356	12.2920	371.967	272.769	99.198	13.527
84.5	3.5354	5.7675	1.7971	11.1000	395.359	303.428	91.931	12.536
86.5	3.1601	5.2038	1.6018	9.9657	416.425	332.209	84.216	11.484
88.5	2.8217	4.6660	1.4411	8.9288	435.319	358.957	76.362	10.413
90.5	2.5214	4.1723	1.3063	8.0000	452.248	382.677	69.571	9.487
92.5	2.2565	3.7278	1.1919	7.1762	467.424	404.937	62.487	8.5210

values are listed in Col's. 2, 3, 4 and 5 of Table IV. The total inflow hydrograph is then divided equally into a number of smaller hydrographs which are assumed to enter the lateral sewer at uniformly spaced points along its length. Each of these local inflow hydrographs are then offset by the time of flow from its point of entry to the lateral outlet. To illustrate the principle, Figure 14 shows the procedure for the case of only five points of entry. The total time of travel from upstream end of the lateral to its outlet is shown as t_r. The summation of the vertical ordinates of all the local offset hydrographs at any time t_o is equal to the vertical ordinate of the outflow hydrograph at t_o. Another way of obtaining the same sum without drawing the local hydrographs is shown in Figure 15. The outflow Q at any time t_o can be computed by summing the instantaneous inflow I at t_o, $(t_o - 1/4\ t_r)$, $(t_o - 1/2\ t_r)$, $(t_o = 3/4\ t_r)$ and $(t_o - t_r)$ and dividing by five. Expressing this mathematically,

$$Q(t_o) = \frac{1}{n} \sum_{i=0}^{i=(n-1)} I\left(t_o - \frac{i}{n-1}\, t_r\right) \tag{29}$$

Multiplying and dividing the equation by $\Delta t_r = \frac{t_r}{n}$

$$Q(t_o) = \frac{\sum_{i=0}^{i=(n-1)} I\left(t_o - \frac{i}{n-1}\, t_r\right) \Delta t_r}{n\, \Delta t_r} \tag{30}$$

If the number of points of entry into the lateral sewer are increased to a large number and Δt_r becomes small, the limit of the sum approaches the integral

$$Q(t_o) = \frac{\int_{t_o - t_r}^{t_o} I(t_o)\, dt_r}{t_r} \tag{31}$$

But this is the expression for the average inflow rate for the period between $(t_o = t_r)$ and t_o. Thus it appears that the value of Q at t_o is equal to the average inflow rate during the period t_r preceding the time t_o, if the number of inflow entry points along the sewer is large.

The time-offset routing method as described herein, gives a fair approximation of the actual outflow hydrograph. As a test of the accuracy of the time-offset method, a series of computations were performed. Hydrographs along a test length of sewer were obtained by the time-offset method and instantaneous rates of flow were taken from these hydrographs and backwater curves computed. Storage values under the backwater curves were determined and plotted against the instantaneous inflow and outflow rates. Hydrographs along the test reach were then computed by the storage routing procedure for a second approximation. This process was again performed giving a third approximation. Comparing the third approximation hydrographs with those first obtained by the time-offset routing, it was found that the time-offset method produced a slightly higher peak rate of flow occurring at a somewhat later time than the more exact method. Because of its simplicity, however, the time-offset method is used for routing the inflow hydrograph through the lateral sewer.

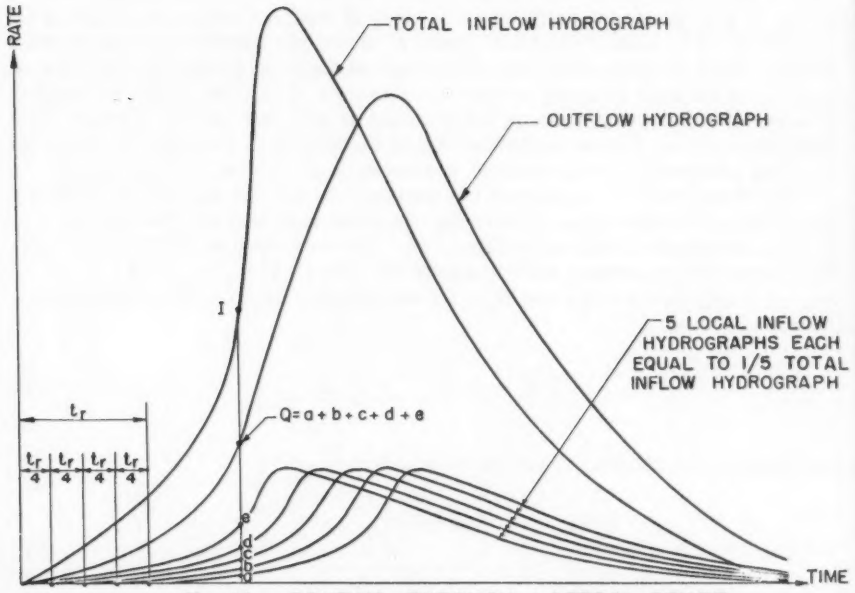


Fig. 14 — ROUTING THROUGH LATERAL SEWER BY OFFSETTING LOCAL HYDROGRAPHS

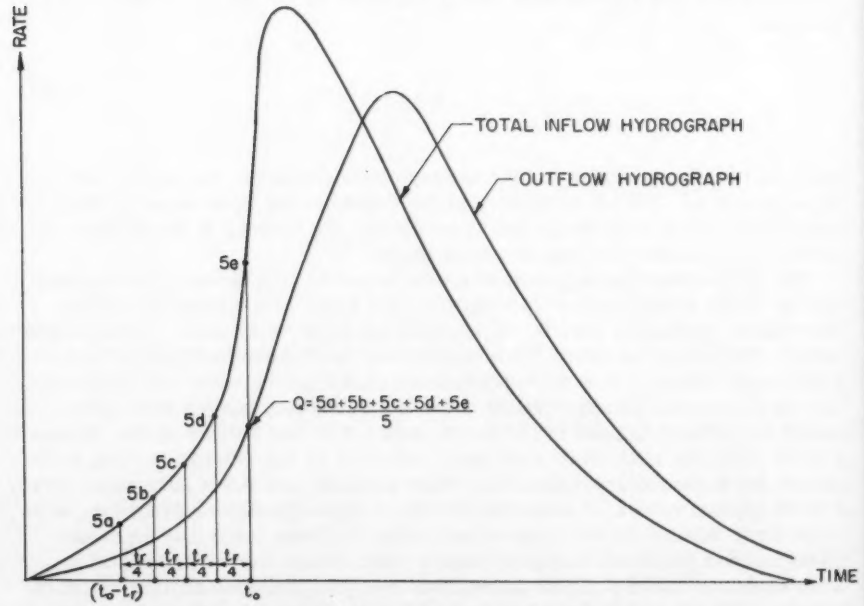


Fig. 15— ROUTING THROUGH LATERAL SEWER BY SUMMATION OF ORDINATES OF TOTAL INFLOW HYDROGRAPH

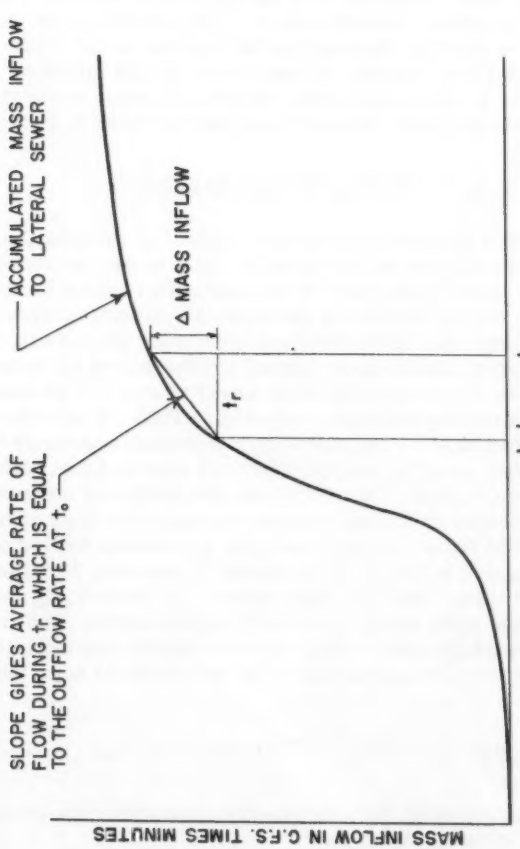

ACCUMULATED MASS INFLOW TO LATERAL SEWER
Δ MASS INFLOW
t_r
SLOPE GIVES AVERAGE RATE OF FLOW DURING t_r WHICH IS EQUAL TO THE OUTFLOW RATE AT t_o
t_o-t_r t_o
TIME IN MINUTES
MASS INFLOW IN C.F.S. TIMES MINUTES

Fig. 16 ROUTING THROUGH LATERAL SEWER BY THE USE OF MASS INFLOW CURVE

The mass inflow, expressed in cubic feet per second times minutes, is tabulated in Column (6), Table IV, and plotted as shown in Figure 16. The mass inflow in the period between (t_o - t_r) and t_o is divided by the time of travel in minutes t_r giving the average rate of inflow in the period t_r, which according to Equation (31) is equal to the outflow Q at the time t_o.

Returning to Table IV, the rate of change in mass I, is derived numerically, as shown in Columns (6), (7) and (8). Dividing the values in Column (8) by 7-1/3 minutes t_r, gives the average inflow rate which, as previously stated equals the outflow rate at t_o. These values are shown in Column (9). Values in Table IV were computed with the aid of an electronic computer, and show greater accuracy than necessary. The time of travel used in this example is found by dividing the length of the lateral sewer (1320') by the average velocity (3 1/sec.) giving 440 seconds or 7-1/3 minutes.

The hydrographs of the outflow from the lateral sewer draining a 10 acre unit area having this and other types of land use is shown in Figure 17.

Step 7 - Routing Through Main Sewer

The sewer pattern as shown in Figure 1, typical of many in the City of Chicago, has the lateral sewers entering the main sewer at uniform intervals. This arrangement lends itself to the same time-offset routing method as outlined for the lateral sewer. In the case of a uniformly developed area of a given type of land use, the hydrograph from each lateral will be almost the same. In any event, the average lateral hydrograph of all lateral hydrographs will be assumed to enter the main sewer at uniform interval. For convenience, the lateral hydrograph computed in Table IV is reduced to the unit rate of c.f.s. per acre. This can be accomplished by merely shifting the decimal point one place to the left, since the flow in c.f.s. represents the runoff from a ten-acre area. Table V shows the method of computation of hydrographs at locations in the main sewer measured by time of travel from the headwater lateral outlet. In this example, the outflow hydrograph of the lateral sewer computed in Table IV, is routed 10 minutes, 30 minutes and 60 minutes time of travel down the main sewer. In Table V, I_m indicates the average inflow to the main sewer from each lateral sewer. The (Mass I_m) can be computed by adding the average rate for the two minute interval and multiplying by two minutes and adding to the accumulated sum, thus

$$.855 = \frac{.041 + .048}{2} \times 2 \text{ min.} + .766$$

The (ΔMass I_m) is computed, for example, by subtracting the (Mass I_m) at (t_o - t_r) from the (Mass I_m) at t_o thus

2.848	@	$t_o = 50.5$
1.444	@	$t_o - 10 = 40.5$
1.404		Δ Mass I_m

The rate of flow Q in c.f.s./acre is then determined by dividing the (Δ Mass I_m) by the time of travel in the main sewer. It will be noted that the runoff rates at any selected time can be determined. It is only necessary to

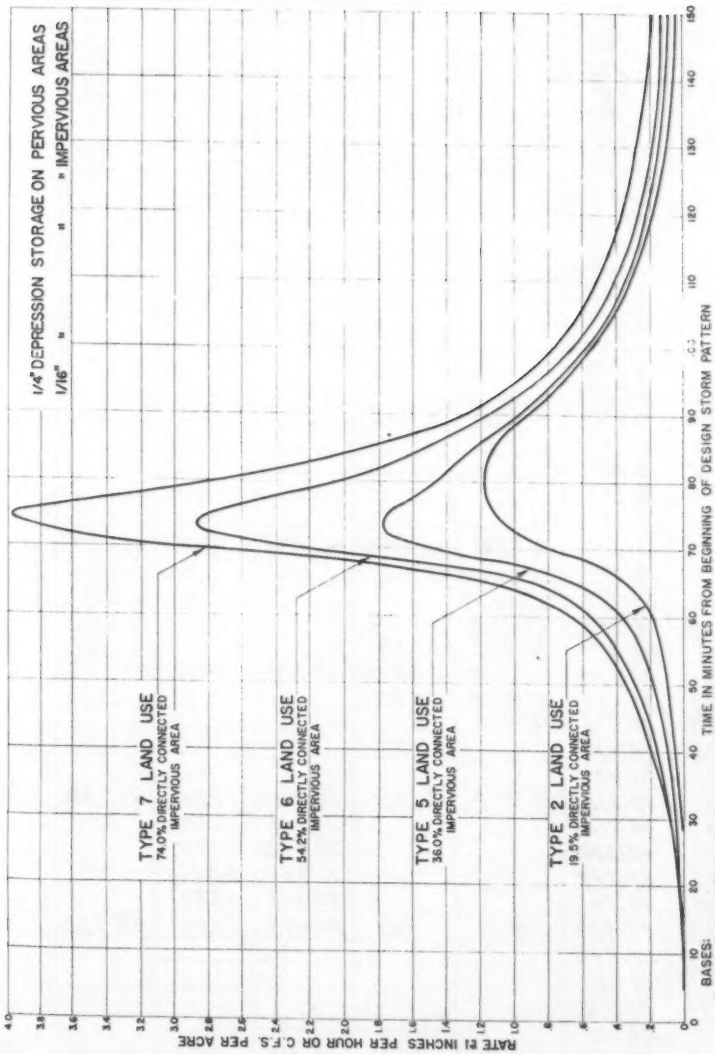

1/4" DEPRESSION STORAGE ON PERVIOUS AREAS
1/16" " " " IMPERVIOUS AREAS
TYPE 7 LAND USE
74.0% DIRECTLY CONNECTED IMPERVIOUS AREA
TYPE 6 LAND USE
54.2% DIRECTLY CONNECTED IMPERVIOUS AREA
TYPE 5 LAND USE
36.0% DIRECTLY CONNECTED IMPERVIOUS AREA
TYPE 2 LAND USE
19.5% DIRECTLY CONNECTED IMPERVIOUS AREA
RATE IN INCHES PER HOUR OR C.F.S. PER ACRE
TIME IN MINUTES FROM BEGINNING OF DESIGN STORM PATTERN
BASES:
(1) TIME-OFFSET ROUTING, VELOCITY = 3 FT./SEC.
(2) DEPRESSION STORAGE AS INDICATED ABOVE

FIG. 17 - EFFECT OF LAND USE ON "10 ACRE" OUTFLOW HYDROGRAPH

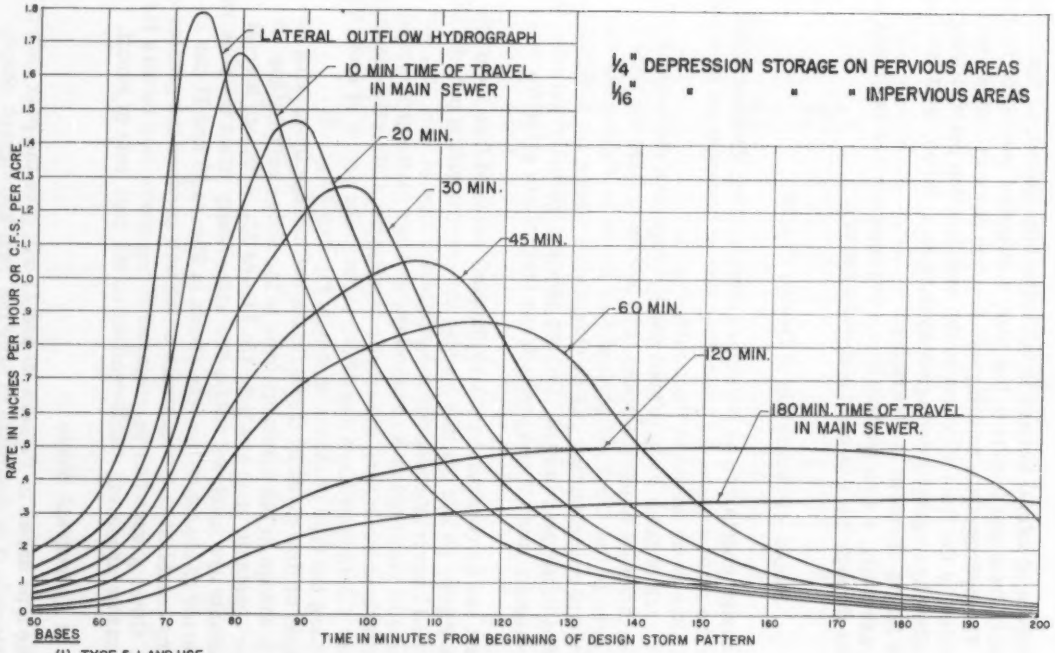
SA 2

TABLE V—ROUTING THROUGH MAIN SEWER

t_o	I_m	Mass I_m	t_r = 10 Min.		t_r = 30 Min.		t_r = 60 Min.	
			Δ Mass I_m	$Q=\frac{\Delta \text{Mass } I_m}{10}$	Δ Mass I_m	$Q=\frac{\Delta \text{Mass } I_m}{30}$	Δ Mass I_m	$Q=\frac{\Delta \text{Mass } I_m}{60}$
Min.	cfs/Acre	(cfs/Acre) Min.	(cfs/Acre) Min.	cfs/Acre	(cfs/Acre) Min.	cfs/Acre	(cfs/Acre) Min.	cfs/Acre
0.00	0*	0 *						
30.5	.041	.766						
32.5	.048	.855						
34.5	.058	.961						
36.5	.073	1.092						
38.5	.088	1.253						
40.5	.103	1.444	.678	.068				
42.5	.118	1.665						
44.5	.131	1.914						
46.5	.146	2.191						
48.5	.162	2.499						
50.5	.187	2.848	1.404	.140				
52.5	.217	3.252						
54.5	.240	3.709						
56.5	.277	4.226						
58.5	.322	4.825						
60.5	.381	5.528	2.680	.268	4.762	.159		
62.5	.472	6.381						
64.5	.607	7.460						
66.5	.856	8.923						
68.5	1.217	10.996						
70.5	1.599	13.811	8.283	.828	12.367	.412		
72.5	1.765	17.175						
74.5	1.782	20.722						
76.5	1.678	24.182	15.259	1.526				
78.5	1.519	27.379	16.383	1.638				
80.5	1.439	30.337	16.526	1.653*	27.489	.916		
82.5	1.353	33.129	15.954	1.595				
84.5	1.254	35.736	15.014	1.501				
86.5	1.148	38.138						
88.5	1.041	40.327						
90.5	.948	42.316	11.979	1.198	36.788	1.226	41.550	.692
92.5	.852	44.116			37.735	1.258		
94.5	.781	45.749			38.289	1.276		
96.5	.713	47.243			38.320	1.277*		
98.5	.645	48.601			37.605	1.254		
100.5	.583	49.829	7.513	.751	36.018	1.201	48.385	.806
102.5	.534	50.946						
104.5	.481	51.961						
106.5	.434	52.876						
108.5	.391	53.701						
110.5	.356	54.448	4.619	.462	24.111	.804	51.600	.860
112.5	.322	55.126					51.874	.865
114.5	.294	55.742					52.033	.867
116.5	.264	56.300					52.074	.868*
118.5	.236	56.800					51.975	.866
120.5	.217	57.253	2.805	.281	14.937	.498	51.725	.862

* Peak Values

* I_m and Mass I_m Between t_o=0 and t_o=30.5 Not Shown


LATERAL OUTFLOW HYDROGRAPH
10 MIN. TIME OF TRAVEL IN MAIN SEWER
20 MIN.
30 MIN.
45 MIN.
60 MIN.
120 MIN.
180 MIN. TIME OF TRAVEL IN MAIN SEWER.
1/4" DEPRESSION STORAGE ON PERVIOUS AREAS
1/16" " " " IMPERVIOUS AREAS
RATE IN INCHES PER HOUR OR C.F.S. PER ACRE
TIME IN MINUTES FROM BEGINNING OF DESIGN STORM PATTERN

BASES

(1) TYPE 5 LAND USE.
(2) LATERAL HYDROGRAPH FROM FIG. 16.
(3) ROUTING BY TIME-OFFSET METHOD.
(4) DEPRESSION STORAGE AS INDICATED ABOVE.

FIG. 18-ROUTING OF LATERAL HYDROGRAPHS ALONG MAIN SEWER

SA 2

choose enough values to be able to plot the curves accurately. If only the peak rates are desired, only these rates near the estimated time of the peak are computed to find the highest value.

In Figure 18, the hydrographs computed in Table V are plotted along with those of other time of travel, for a uniformly developed area using Type 5 Land Use.

Where the pattern of lateral sewers is not uniformly spaced, or the Land Use varies greatly from one lateral to the next, a different method of routing must be used. The hydrographs entering one junction point are added together, then shifted by the time of travel to the next junction point, adding the new local inflow hydrographs and again shifting, and so on, until the flow along the entire length of sewer is determined.

In practice, generally, a combination of the two methods are employed in routing the flows through the sub-main and main sewers.

Step 8 - Preparation of Design Charts

Convenient design charts can be developed after routing the lateral hydrographs through the main sewers and producing sets of hydrographs at various times of travel, like those shown in Figure 18.

The peaks of each of these hydrographs and hydrographs similarly routed for areas having different imperviousness, are plotted against the time of concentration, used in the "Rational Method," as shown in Figure 19. The approximate time of concentration can be obtained by adding ten minutes to the time of travel in the main sewer. Curves showing intensity-duration data for a 5 year frequency rainfall and percentages thereof are given for comparison.

Another chart can be obtained by taking the peak rates of flow from the hydrographs in Figure 18 and plotting against the corresponding percent of directly connected imperviousness for the particular type of land use, and the time of travel is recorded as a parameter. Similar plottings are made for other types of land use having different percentages of directly connected imperviousness. Continuous lines are drawn through the points of equal time of travel producing the type of charts shown in Figures 20 and 21.

The peak rates of flow given by these charts will change by varying degrees with each change in the constants used in the above seven steps. A summary of the constants used are listed in Figure 22. Although a great number of constants are involved, only a few will greatly affect the peak rate of runoff. A series of charts of the type shown in Figure 20 and 21 can be prepared for any combination of constants which will produce the greater effect on runoff. Other graphs can be prepared giving correction factors for changes in constants producing smaller effect on the peak rate of runoff.

Determination of Hydrograph Shape

Charts of the type shown in Figures 20 and 21 will give the peak rate of flow, but occasionally it is desirable to have the total hydrograph shape, such as for pumping station design, for branch sewers, etc. For fairly uniform drainage areas, a chart as shown in Figure 23 is prepared to facilitate the computation of the hydrograph for any time of travel and for areas having any imperviousness. This chart is merely the accumulated mass of the lateral hydrographs, from unit-areas of varying imperviousness, similar to

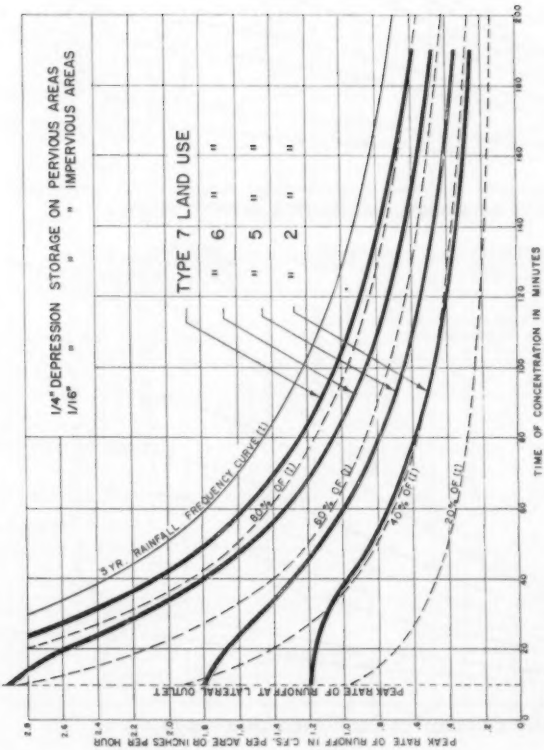
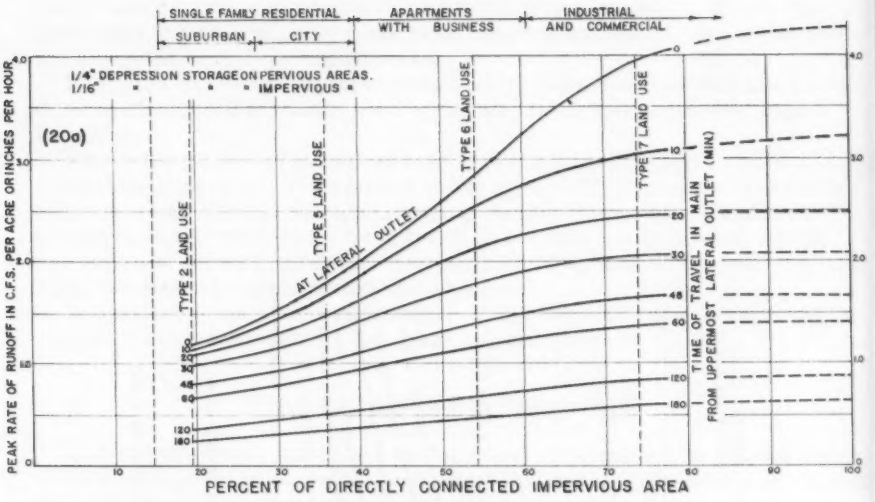
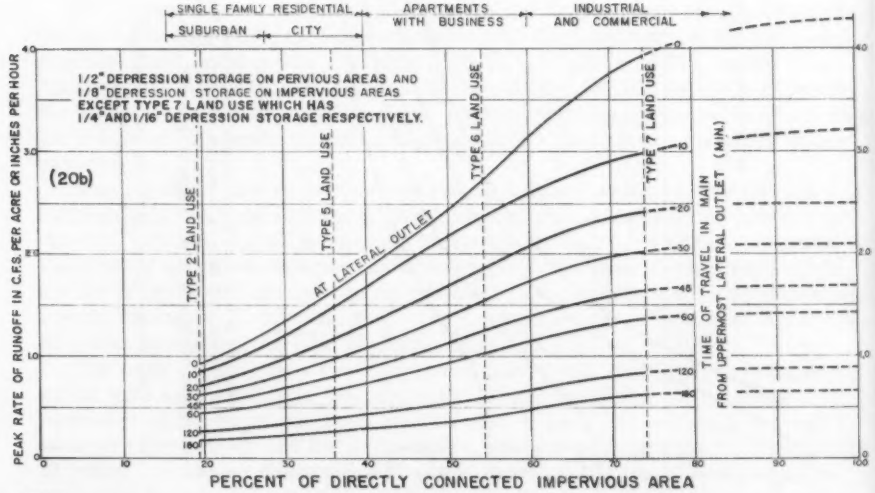

1/4" DEPRESSION STORAGE ON PERVIOUS AREAS
1/16" " " IMPERVIOUS AREAS
TYPE 7 LAND USE
" 6 " "
" 5 " "
" 2 " "
3 YR RAINFALL FREQUENCY CURVE (1)
80% OF (1)
60% OF (1)
40% OF (1)
20% OF (1)
PEAK RATE OF RUNOFF AT LATERAL OUTLET
PEAK RATE OF RUNOFF IN C.F.S. PER ACRE OR INCHES PER HOUR
TIME OF CONCENTRATION IN MINUTES
BASES:
(1) HYDROGRAPH METHOD USING "5 YEAR" DESIGN STORM PATTERN.
(2) PEAK RATES DERIVED FROM FIG 18 AND SIMILAR CHARTS FOR OTHER TYPES OF LAND USE.
(3) TIME OF CONCENTRATION ASSUMED EQUAL TO 10 MINUTES PLUS THE TIME OF TRAVEL FROM OUTLET OF EXTREME HEADWATER LATERAL.

Fig. 19—EFFECT OF LAND USE ON PEAK RUNOFF RATES AND THEIR RELATION TO RAINFALL RATES

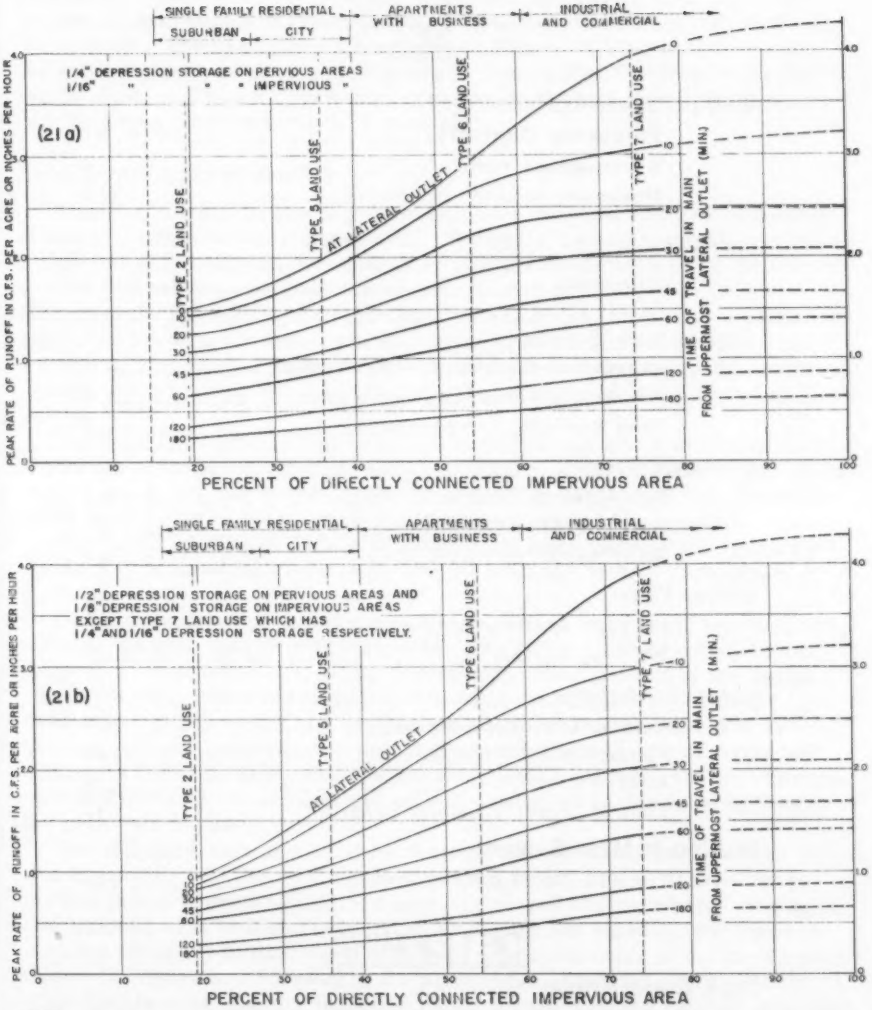

SINGLE FAMILY RESIDENTIAL
SUBURBAN
CITY
APARTMENTS WITH BUSINESS
INDUSTRIAL AND COMMERCIAL
1/4" DEPRESSION STORAGE ON PERVIOUS AREAS.
1/16" " " " IMPERVIOUS "
(20a)
TYPE 2 LAND USE
TYPE 5 LAND USE
TYPE 6 LAND USE
TYPE 7 LAND USE
AT LATERAL OUTLET
TIME OF TRAVEL IN MAIN FROM UPPERMOST LATERAL OUTLET (MIN.)
PEAK RATE OF RUNOFF IN C.F.S. PER ACRE OR INCHES PER HOUR
PERCENT OF DIRECTLY CONNECTED IMPERVIOUS AREA


SINGLE FAMILY RESIDENTIAL
SUBURBAN
CITY
APARTMENTS WITH BUSINESS
INDUSTRIAL AND COMMERCIAL
1/2" DEPRESSION STORAGE ON PERVIOUS AREAS AND
1/8" DEPRESSION STORAGE ON IMPERVIOUS AREAS
EXCEPT TYPE 7 LAND USE WHICH HAS
1/4" AND 1/16" DEPRESSION STORAGE RESPECTIVELY.
(20b)
TYPE 2 LAND USE
TYPE 5 LAND USE
TYPE 6 LAND USE
TYPE 7 LAND USE
AT LATERAL OUTLET
TIME OF TRAVEL IN MAIN FROM UPPERMOST LATERAL OUTLET (MIN.)
PEAK RATE OF RUNOFF IN C.F.S. PER ACRE OR INCHES PER HOUR
PERCENT OF DIRECTLY CONNECTED IMPERVIOUS AREA

BASES

(1) HYDROGRAPH METHOD USING "5 YEAR" DESIGN STORM PATTERN.
(2) STANDARD INFILTRATION CURVE, FOR TYPE 2, 5, & 6 LAND USE, AS SHOWN IN FIG. 4
REDUCED INFILTRATION CURVE, FOR TYPE 7 LAND USE.
(3) RATES AT LATERAL OUTLET, TAKEN FROM HYDROGRAPHS IN FIG. 17
(4) RATES AT OTHER TRAVEL TIMES DOWNSTREAM TAKEN FROM FIG. 18
AND SIMILAR CHARTS FOR OTHER TYPES OF LAND USE.
(5) DEPRESSION STORAGE AS INDICATED ABOVE.

FIG. 20— RUNOFF RATES IN SEWER SYSTEM VS. TRAVEL TIME AND PERCENT OF DIRECTLY CONNECTED IMPERVIOUS AREAS— AVERAGE GROUND SLOPE = 0.01


SINGLE FAMILY RESIDENTIAL
SUBURBAN
CITY
APARTMENTS WITH BUSINESS
INDUSTRIAL AND COMMERCIAL
1/4" DEPRESSION STORAGE ON PERVIOUS AREAS
1/16" " " " IMPERVIOUS "
(21a)
TYPE 2 LAND USE
TYPE 5 LAND USE
AT LATERAL OUTLET
TYPE 6 LAND USE
TYPE 7 LAND USE
TIME OF TRAVEL IN MAIN FROM UPPERMOST LATERAL OUTLET (MIN.)
PEAK RATE OF RUNOFF IN C.F.S. PER ACRE OR INCHES PER HOUR
PERCENT OF DIRECTLY CONNECTED IMPERVIOUS AREA
1/2" DEPRESSION STORAGE ON PERVIOUS AREAS AND 1/8" DEPRESSION STORAGE ON IMPERVIOUS AREAS EXCEPT TYPE 7 LAND USE WHICH HAS 1/4" AND 1/16" DEPRESSION STORAGE RESPECTIVELY.
(21b)

BASES

(1) HYDROGRAPH METHOD USING "5 YEAR" DESIGN STORM PATTERN.
(2) STANDARD INFILTRATION CURVE, FOR TYPE 2, 5, & 6 LAND USE, AS SHOWN IN FIG. 4 REDUCED INFILTRATION CURVE, FOR TYPE 7 LAND USE.
(3) RATES AT LATERAL OUTLET, TAKEN FROM HYDROGRAPHS IN FIG. 17
(4) RATES AT OTHER TRAVEL TIMES DOWNSTREAM TAKEN FROM FIG. 18 AND SIMILAR CHARTS FOR OTHER TYPES OF LAND USE
(5) DEPRESSION STORAGE AS INDICATED ABOVE.

FIG. 21 — RUNOFF RATES IN SEWER SYSTEM VS. TRAVEL TIME AND PERCENT OF DIRECTLY CONNECTED IMPERVIOUS AREAS — AVERAGE GROUND SLOPE = 0.02

SET

Design Storm Pattern:

Frequency Constants ..... a, b, & c
Advanceness Ratio ..... r
Maximum Duration of Rainfall 180

Infiltration:

Constants ..... f_o, f_c & k

Depression Storage:

Impervious Surface, Average Depth ..... d
Pervious Surface, Average Depth ..... d
Roof Surface, Average Depth ..... d

Overland Flow:

Pavement Constants ..... ℓ, s & c
Pervious Area Constants ..... ℓ, s & c
Roof Area Constants ..... ℓ, s & c

Gutter Flow:

1st Type Gutter ..... L_g, S_g, Z, n, a_1, & a_2
2nd Type Gutter ..... L_g, S_g, Z, n, a_1, & a_2

Lateral Routing:

Number of 1st Type Gutter 24
Number of 2nd Type Gutter 8
Area of Roofs ..... a_3
Time of Travel in Lateral Sewer ..... t_r

Routing In Main Sewer:

Time of Travel in Main Sewer ..... t_r

DETERMINE

Peak Runoff Rate:

cfs/acre ..... Q

Reduction for Areal Distribution of Rainfall:

For Area Over 640 Acres. ..... KQ

Fig.22-LIST OF CONSTANTS USED TO DETERMINE PEAK RATE OF RUNOFF.

the tabular values shown in Column (3) Table V for Type 5 Land Use. The method of determining the complete hydrograph is identical to that outlined for Table V. The hydrographs computed by Figure 23 are consistent with the values of peak rates of runoff shown in Figure 20a. Changes in the constants shown in Figure 22 will cause changes in the unit-area hydrographs and therefore affect changes in Figure 23.

Areal Distribution of Rainfall

Thus far, the design storm pattern has been assumed to occur uniformly at every location of the drainage area. Normally, however, the storms have a high intensity core and decrease in intensity with distance from the storm center. A chart reproduced from an unpublished report by the late Mr. Eltinge of the Sanitary District of Chicago, Figure 24, shows the reduction of rainfall rates with distance from the center of storm for various duration rainfalls. The rainfall frequency curve used for deriving the design storm pattern gives the average rate of rainfall to be expected at the center of the storm.

The right hand side of Figure 25, derived from the graph in Figure 24, shows the average intensity of rainfall as a ratio of the intensity at the storm center, for a circular shaped drainage area. The duration of the rainfall is taken as the time of travel from the lateral outlet plus ten minutes.

The center portion of Figure 25 gives guide lines showing additional reductions to be used for rectangular shaped drainage area of various proportions.

If the average rainfall over the drainage area is reduced by a factor, how much is the peak rate of runoff reduced? To answer this question, runoff rates were computed using ninety percent and eighty percent of the design storm pattern. The fixed values of infiltration and depression storage effectively reduces the runoff to a greater extent on pervious areas. For one hundred percent impervious areas, it is assumed that the runoff rates will reduce by the same percentage as the reduction in rainfall. On the left hand side of Figure 25, guide lines will give the reduction factor to be applied to the peak rate of runoff.

The tributary area and the time of travel for any reach in the sewer system is entered into the right hand side of the chart, Figure 25, and the reduction factor is determined for a circular shaped drainage area. A map of the drainage area is then inspected to determine the approximate shape of the area tributary to that reach of sewer. The guide lines in the center portion of the figure are followed to the point of the estimated width to length of the drainage area, and the reduction factor for the average rainfall rate over the tributary drainage area is obtained. Applying this reduction factor to the chart on the left hand side of Figure 25 and following the guide lines to the imperviousness of the tributary drainage area, the reduction factor to be applied to the peak rate of runoff is determined.

Sample of Computation Method for Small Drainage Areas

Figure 26 shows a layout of a proposed sewer system for the conveyance of storm water. It also shows a profile of the ground surface along the principal sewer line. Below the ground surface is indicated the desirable

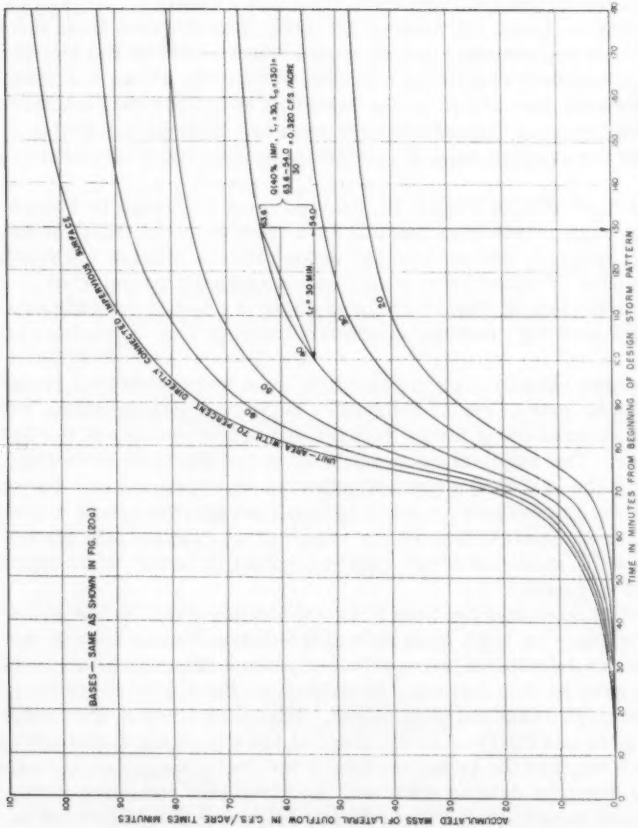

ACCUMULATED MASS OF LATERAL OUTFLOW IN C.F.S./ACRE TIMES MINUTES
TIME IN MINUTES FROM BEGINNING OF DESIGN STORM PATTERN
BASES — SAME AS SHOWN IN FIG. (20a)
UNIT-AREA WITH 70 PERCENT DIRECTLY CONNECTED IMPERVIOUS SURFACES
0(40% IMP, t_c = 30, t_d = 130)
$\frac{516-340}{30}$ + 0.320 CFS /ACRE
516
340
t_c = 30 MIN
20
40
60
80

FIG. 23—CHART USED TO DETERMINE HYDROGRAPH SHAPE FOR UNIFORMLY DEVELOPED AREAS

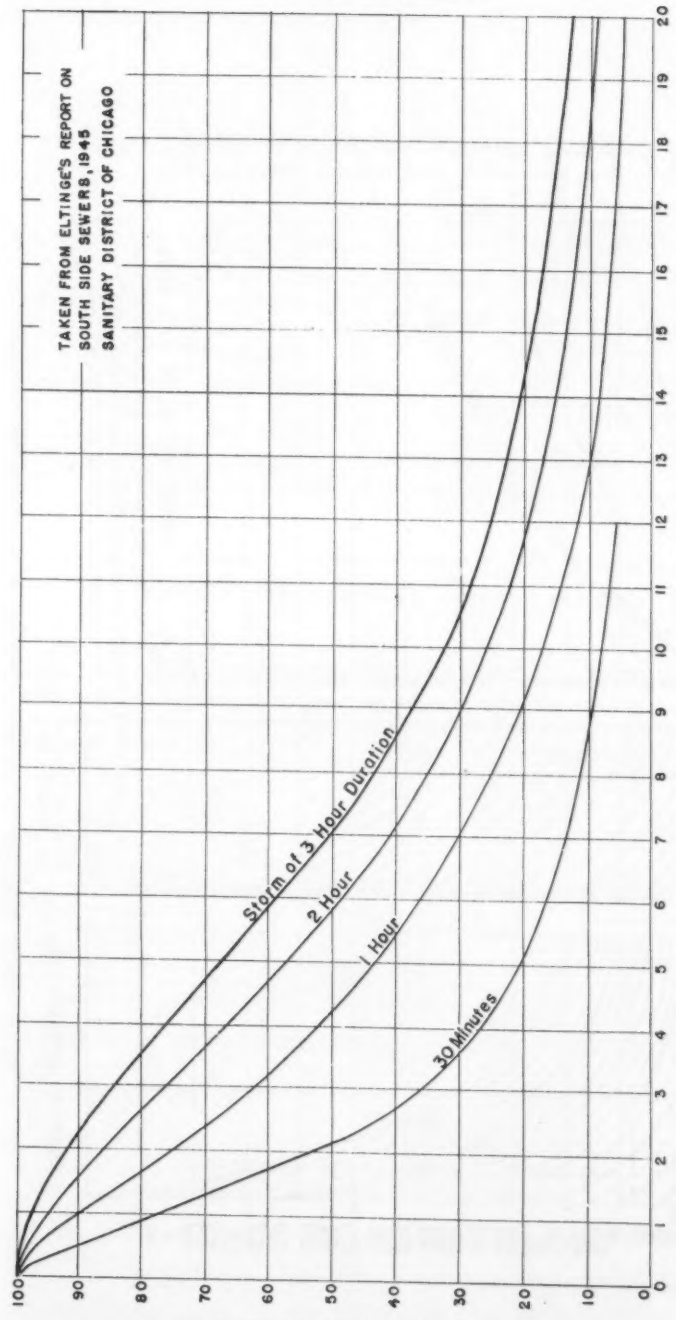


Fig. 24 – VARIATION OF RAINFALL INTENSITY WITH DISTANCE FROM CENTER OF STORM

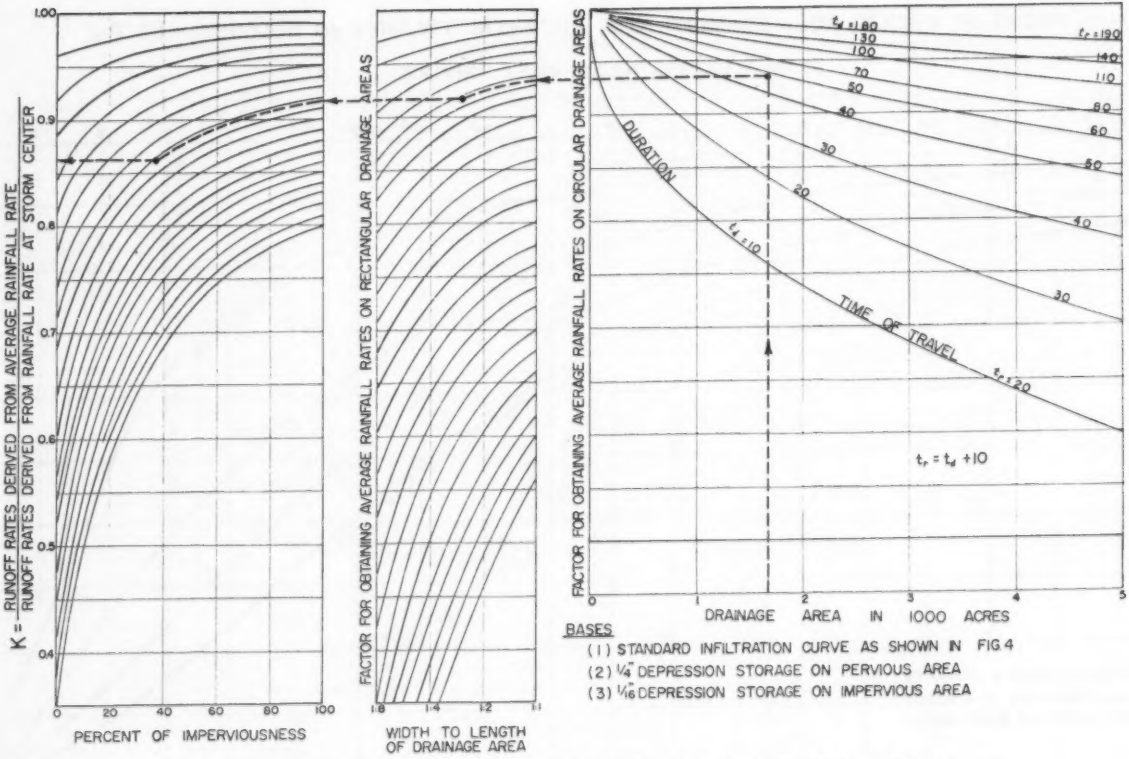
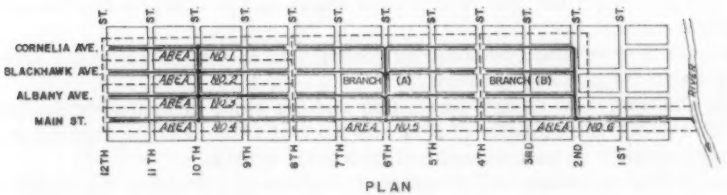

K = RUNOFF RATES DERIVED FROM AVERAGE RAINFALL RATE / RUNOFF RATES DERIVED FROM RAINFALL RATE AT STORM CENTER
PERCENT OF IMPERVIOUSNESS
FACTOR FOR OBTAINING AVERAGE RAINFALL RATES ON RECTANGULAR DRAINAGE AREAS
WIDTH TO LENGTH OF DRAINAGE AREA
FACTOR FOR OBTAINING AVERAGE RAINFALL RATES ON CIRCULAR DRAINAGE AREAS
DURATION
TIME OF TRAVEL
$t_r = t_d + 10$
DRAINAGE AREA IN 1000 ACRES
BASES
(1) STANDARD INFILTRATION CURVE AS SHOWN IN FIG 4
(2) 1/4" DEPRESSION STORAGE ON PERVIOUS AREA
(3) 1/16" DEPRESSION STORAGE ON IMPERVIOUS AREA

Fig. 25 — RUNOFF REDUCTION FACTOR ACCOUNTING FOR AREAL DISTRIBUTION OF RAINFALL

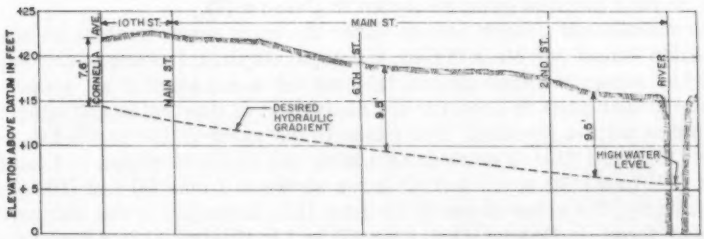
TABLE VI – COMPUTATION FORM FOR SMALL DRAINAGE AREA
(Illustrative Example with Reference to Fig. 20(a))

LINE	LOCATION		Length of Reach	Slope of Hyd. Grad.	Velocity in Reach	TIME OF TRAVEL		AREA OF ADDITION		ΣA	IMPERVIOUSNESS				PEAK FLOWS	
	IN	AT				in Reach	To	No.	A		I_p	$A \times I_p$	$\Sigma A I_p$	I_p for ΣA	Unit Rate	Total
			Ft	Ft/Ft	Ft/Sec.	Min.	Min.		Ac.	Ac.	%	Ac	Ac	%	CFS/Ac	CFS
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1		CORNELIA AV.					0	(1)	20	20	38.6	772	772	38.6	1.93	39
2	TENTH ST.		330	.002	4.6	1.2										
3		BLACKHAWK					1.2	(2)	20	40	34.5	690	1462	36.5	1.80	72
4	"		330	.0017	5.0	1.1										
5		ALBANY AV.					2.3	(3)	20	60	36.0	720	2182	36.4	1.76	107
6	"		330	.0014	5.1	1.1										
7		MAIN ST.					3.4	(4)	20	80	45.5	910	3092	38.6	1.88	150
8	MAIN ST.		2640	.0012	5.3	8.3										
9		SIXTH ST.						(5)	20		42.8	856				
10		SIXTH ST.					11.7	Bk. A	60	160	35.2	2112	5060	37.9	1.75	280
11	"		2640	.001	5.7	7.7										
12		SECOND ST.						(6)	23		56.1	1152				
13		SECOND ST.					19.4	Bk. B	34	217	37.2	1268	8480	39.0	1.59	345
14	"		1650	.0009	5.8	4.7										
15		RIVER					24.1									345

LAYOUT OF PROPOSED SEWER SYSTEM



PLAN



PROFILE

FIG. 26–ILLUSTRATIVE EXAMPLE FOR SMALL DRAINAGE AREA

Fig. 25 – RUNOFF REDUCTION FACTOR ACCOUNTING FOR AREAL DISTRIBUTION OF RAINFALL

hydraulic gradient for the drainage of basements in the area. This dotted line establishes the available slope of the energy gradient for use in determining flow velocities, time of travel and sewer sizes.

Table VI is a sample computation for the determination of design flows as listed in Column (16). It has been previously determined, that the design storm is as shown in Figure 3, that the average ground slope of the pervious areas is about 0.01 and it has been estimated that the character of the surface is such as to retain 1/4" of water in the form of depression storage on pervious areas and 1/16" on impervious areas. With those assumptions and others listed in Figure 20a the curves shown in that figure are accepted as a basis of design. Previous studies have determined the portions of the drainage area and sub-areas which are covered by directly connected impervious surfaces, based upon the degree of development and type of land use assumed for design.

Data for Columns (1) to (4) are taken from Figure 26. The computations then proceed as follows:

Columns (8) to (14) show the size of areas and the derivations of the percent of imperviousness for such areas tributary to the street intersections indicated in Columns (1) and (2). By multiplying the acreage of each sub-drainage area by the percent of imperviousness of that sub-area, adding the products and then dividing the cumulative products by the cumulative area, the percent of imperviousness of the cumulative area is derived and listed in Column (14).

By using the value of imperviousness from Column (14) and the time of travel in Column (7) the unit rate of flow shown in Column (15) is derived from Figure 20a.

The total flow is then computed by multiplying the unit rate by the cumulative area to the point in question.

Using the total flow from Column (16) and the slope of hydraulic gradient from Column (4), the velocity of flow and time of travel can be derived in the customary manner from conventional hydraulic charts.

Time of flow as shown in Column (6) is obtained by dividing the length of reach by velocity of flow in Column (5) cumulative time of flow is then recorded at each junction point as shown in Column (7).

Where additional branch sewers enter the main sewer such as at the corner of Sixth Street and Main Street, the separate drainage areas are separately listed as shown in Lines (9) and (10) and the summation of all areas tributary to that point is used for the derivation of flow rates and time of travel below such a junction. For example the value of 160 acres for Column (10), Line (10) is derived by adding the 80 acres shown in Line (7), Column (10) to the 20 acres and 60 acres shown in Lines (9) and (10), Column (9). Similarly, the value shown in Column (13), Line (10) is the summation of the values shown in Column (13), Line (7) and in Column (12), Lines (9) and (10).

These computations indicate the simplicity of design for small or moderately sized sewer systems by the use of the Hydrograph Method.

Sample of Computation Method for Large Drainage Areas

Applying the hydrograph method to a large drainage area will illustrate the use of the charts developed in Step (8). Rainfall reduction factors and

FIG. 27 – LAYOUT OF LARGE DRAINAGE AREA AND HYDROGRAPHS AT JUNCTION POINTS

TABLE VII - COMPUTATION FORM FOR A LARGE DRAINAGE AREA

(Illustrative Example with reference to Figures 18, 20(a), 23, 25, 27, and 28.)

LINE	LOCATION		LENGTH OF REACH	SLOPE OF HYD GRAD	VELOCITY IN REACH	TIME OF TRAVEL Tp		AREA OF ADDITION		ΣA	IMPERVIOUSNESS				PEAK FLOWS				INSTANTANEOUS FLOWS		
	IN	AT				IN REACH	TO	NO.	A		Ip	Ip x A	Σ Ip x A	Ip^0.6 x ΣA	q	Q = qA	K	Qp = KQ	KQ75	KQ80	KQ95
			ft.	ft./ft.	ft./sec.	Min.	Min.		Ac.	Ac.	%	Ac.	Ac.	%	cfs/Ac.	C.F.S.	%	C.F.S.	C.F.S.	C.F.S.	C.F.S.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
1		184+80					0	A-1	20	20	36	7.20	7.20	36	1.792	36	100	36	36	29	15
2	WASHINGTON		330	.0028	5.05	1.09															
3		181+50					1.09	A-2	20	40	36	7.20	14.40	36	1.782	71	100	71	71	59	32
4	"		330	.0017	4.95	1.11															
5		178+20					2.20	A-3	20	60	36	7.20	21.60	36	1.772	106	100	106	106	90	49
6	"		330	.0014	5.10	1.08															
7		174+90					3.28	A-4	20	80	36	7.20	28.80	36	1.760	141	100	141	140	121	66
8	"		330	.0011	5.00	1.10															
9		171+60					4.38	A-5	40	120	36	14.40	43.20	36	1.715	205	100	205	200	180	102
10	"		660	.00084	5.00	2.20															
11		165+00					6.58	A-6	40	160	36	14.40	57.60	36	1.690	270	100	270	255	245	145
12	"		660	.00073	4.97	2.21															
13		158+40					8.79	A-7	40	200	36	14.40	72.00	36	1.672	335	100	335	300	320	190
14	"		660	.00063	4.97	2.21															
15		151+80					11.00	A-8	40	240	36	14.40	86.40	36	1.668	400	100	400	335	400	244
16	"		660	.00058	5.00	2.20															
17		145+20					13.20	A-9	80	320	36	28.80	115.20	36	1.560	500	100	500	365	480	350
18	"		1320	.00048	4.98	4.41															
19		132+00					17.61	A-10	80	400	36	28.80	144.00	36	1.450	580	100	580	384	522	455
20	"		1320	.00043	5.00	4.40															
21		118+80					22.01	A-11	80	480	36	28.80	172.80	36	1.425	685	100	685	396	540	612
22	"		1320	.00040	5.02	4.38															
23		105+60					26.39	A-12	80	560	36	28.80	201.60	36	1.335	748	100	748	406	550	736
24	"		1320	.00038	5.03	4.37															
25		92+40					30.76	A-13	40	600	36	14.40	216.00	36	1.268	760	100	760	384	520	760
26	"		1320	.00035	4.97	4.41															
27		16TH ST.					35.17	A-14	40	640	36	14.40	230.40	36	1.191	761	89	678	310	427	660
28	16TH ST.		2640	.0017	8.80	5.00															
29		MAIN ST.					40.17	B	400	1040	38	152.00	382.40	36.7	Computed		86.5	1058	467	710	1052
30	MAIN ST.		2640	.0004	5.50	8.00															
31		8TH ST.						C	240	1280	40	96.00	478.40		by Plotting						
32		"					48.17	D	80	1360	40	32.00	510.40	37.5			86.2	1268	484	706	1230
33	"		2640	.0004	5.80	7.60									and Shifting						
34		RIVER RD.						E	240	1600	45	108.00	618.40								
35		"					55.77	F	80	1680	50	40.00	658.40	38.1	Hydrographs		86.2	1445	698	924	1332
36	"		660	.00035	5.60	1.90									→	←		1445	698	924	1332
37		OUTFALL					57.67														

instantaneous backwaters are applied for the determination of required sewer sizes and gradients.

A typical layout of the drainage area to be studied is shown in Figure 27. An examination is made of the various sub-areas to determine their area and imperviousness. Routes along which the main sewer and branch lines are to be built are selected so as to take advantage of the natural ground slopes. Profiles are drawn along the selected routes and as illustrated in Figure 26, the maximum permissible hydraulic gradient lines are established to adequately drain basements in the area. Computation forms such as those shown in Tables VI and VII are then prepared. For the branch sewers or for drainage areas nearly uniformly developed, charts like those shown in Figures 20 and 21 can be used to determine the peak rate of flow, and with the aid of Figure 23 the entire hydrograph at any location can be determined.

The computation procedure for area "A," illustrated in Table VII, Columns (1) to (16) and Lines (1) to (27) is similar to that outlined for Table VI for the small drainage areas. Because the total drainage area substantially exceeds the 640 acre limit for use of uniform areal distribution of rainfall, Columns (17) and (18) are added to Table VII to apply the areal-distribution reduction factor "K" as presented in Figure 25. The factor of unity is used for any area less than 640 acres. Columns (19), (20) and (21) are used to record the instantaneous discharge rates taken from a set of hydrographs like those shown in Figure 18. For example, in Line (25) the time of travel is 30.76 minutes, the area is 600 acres and the imperviousness is 36 percent. From the hydrograph in Figure 18 with time of travel slightly longer than 30 minutes, the rate at 80 minutes after beginning of rainfall is 0.87 c.f.s. per acre. Multiplying 0.87 c.f.s. per acre by 600 acres and "K" equals 1.0 gives 520 c.f.s. This is recorded in Column (20).

Similar computations, as shown in Table VII, Lines (1) to (27), are performed for every branch sewer.

A small table in Figure 27 shows the tributary area, the average imperviousness and the time of travel for each of the branch sewers. With the values given in this table and with the use of the mass curves in Figure 23, the hydrographs of flow into the main sewer can be computed. The branch and local inflow hydrographs at each of the function points are shown at the bottom of Figure 27.

The branch and local area hydrographs are routed through the main sewer by offsetting the hydrographs by the time of travel between junction points. The hydrograph at Junction Point (1) is offset by 5.0 minutes and added to the Branch "B" Hydrograph giving the outflow hydrograph at Junction Point (2). The outflow hydrograph at Point (2) is then shifted by 8.0 minutes and added to the Branch "C" and Local Area "D" Hydrographs, producing the outflow hydrograph at Junction Point (3). The outflow hydrograph at Junction Point (3) is shifted by 7.6 minutes to Point (4) and added to the Branch "E" and Local Area "F" Hydrographs producing the outflow hydrograph at Junction Point (4).

Simultaneous with the development of the main sewers hydrographs, entries are made in Table VII, for time of travel in the reach downstream of the junction point, the tributary area, average imperviousness and the "K" factor. The peak rates and the rates at instantaneous times are taken from the main sewer hydrographs and are multiplied by the "K" factor and entered into Columns (18) to (21). Preliminary size and profile of the main sewer is based on the peak flow from Column (18).

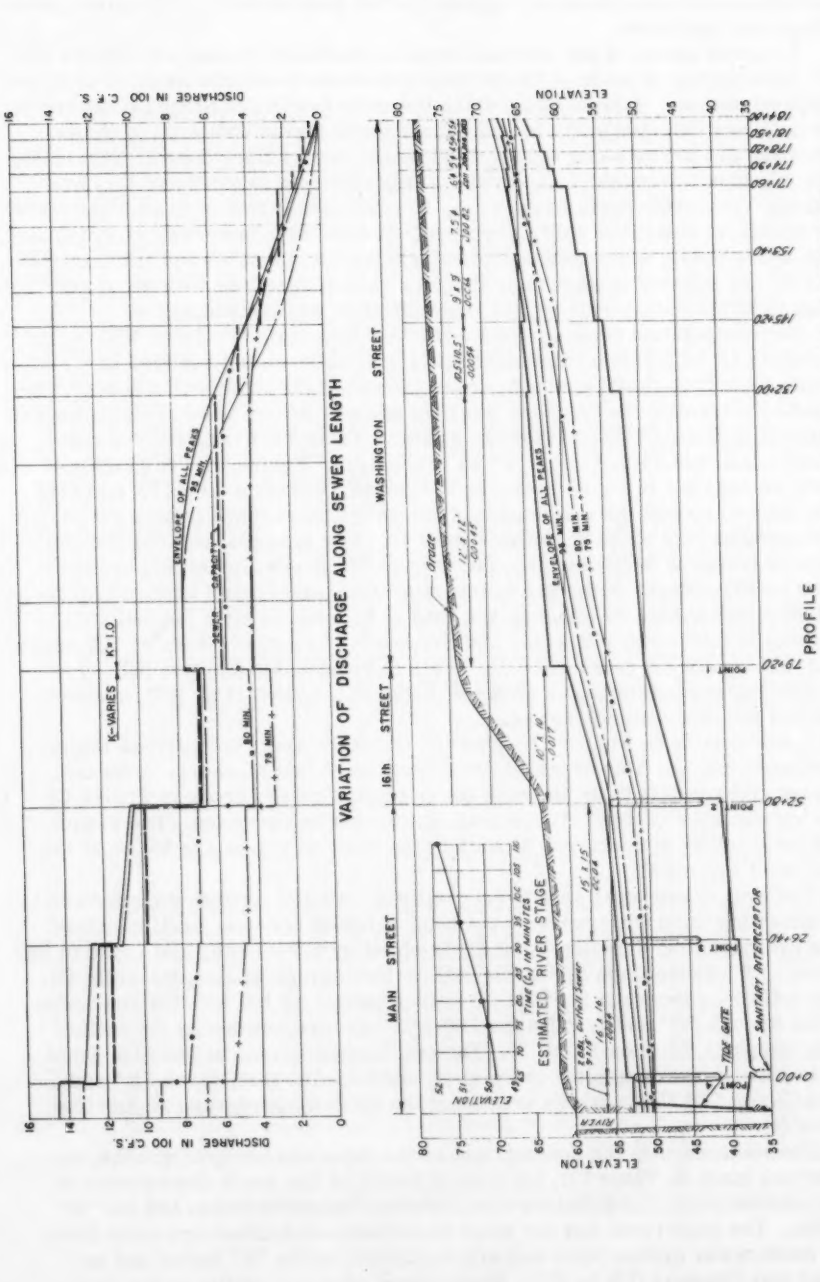


FIG. 28 — INSTANTANEOUS DISCHARGES AND WATER SURFACE PROFILES ALONG MAIN SEWER

A profile along the main line and Branch "A" is shown in Figure 28. At the top of Figure 28, the instantaneous rates and the envelope of all the peak rates are plotted graphically. Shown for comparison, are the capacities of the sewers flowing full with friction loss equal to invert slope. Instantaneous backwater curves were computed and are shown on the profile along with a backwater curve using the envelope of all the peak discharges. The envelope backwater was computed cognizant of the fact that it is not a true relationship at any one time, since the loss of energy at the time of occurrence of the downstream peak cannot be directly added to the losses obtained for the upstream peaks occurring at an earlier time. Before starting the backwater curves, the downstream control must be determined. If the flow enters a river having variable water surface, the stage of the river must be estimated for the time of the instantaneous backwater curve.

An examination of the instantaneous backwater curves indicate the travel of the flood wave along the sewer. Often, after examination of the instantaneous backwater profiles, the preliminary sizing of sewer can be adjusted downward, resulting in savings in sewer costs.

FIG. 28 — INSTANTANEOUS DISCHARGES AND WATER SURFACE PROFILES ALONG MAIN SEWER

CONCLUSION

After a series of charts such as those in Figures 20 and 21 are prepared, the ease of determining the peak runoff rate is equal to that of using the "Rational Method." With the advent of the electronic computing machine, it becomes feasible to prepare many charts of this type covering the complete range of those parameters which principally affect the peak rates of runoff and to prepare correction charts for parameters causing minor changes in the peak rates of runoff. In selecting the proper chart, the engineer must consider the physical aspects of the drainage area such as: type of soil so as to determine the proper infiltration curve; estimate the amount of depression storage; the slope, length and roughness coefficients of the average overland flow strips; and other measurable quantities. An estimation of the value to be assigned to a number of individual parameters should produce more logical results than the selection of a single all-embracing coefficient such as used in the "Rational Method."

Although a great deal of time has been spent in the development of the methods outlined herein, the authors believe each step can and should be improved. Further studies are being made on more exact storage routing procedures to be used in the lateral and main sewers.

An examination of Figure 9 reveals the large effect of depression storage upon the runoff rates from elemental strips of pervious area. Knowledge of depression storage, therefore, is very important in sewer design for open residential or suburban areas. More research is desirable to determine the amounts of depression storage to be used for design.

Ground slope has little effect upon sewer design for densely populated urban drainage areas, since, in such areas, the delaying effect of overland flow causes the runoff from these surfaces to reach the sewer well after the peak runoff from the roofs and pavements.

Roof slopes are not related to ground slopes, and pavement slopes are inter-related with spacing of street inlets so as to produce compensating storage volumes in the gutter channels. In suburban areas, on the other hand, the proportion of directly connected impervious areas is less and runoff from

ground surfaces may be the principal influence upon peak flows. Ground slope, therefore, becomes more important. A comparison of values shown in Figure 20 and 21 will illustrate this point.

Other abstractions and reducing factors that should be considered are, evaporation, interception by trees and shrubs, wetting of surfaces, leakage into and out of house drains.

This paper does not purport to be a final diagnosis of the intricate relationships between rainfall and runoff in an urban area. It is hoped by the authors, however, that it may, in some measure, guide the thinking of the sewer designer to a clearer visualization of what happens, hydrologically, when rain falls on the heterogeneous catchment surfaces of a city. From such an improved visualization should emerge more logical sewer design and more efficient and economical sewer systems.

List of Symbols

A	Drainage area in Acres.
A	Cross-section area of gutter in square feet.
a	Constant used in rainfall rate-duration-frequency curve.
b	Constant used in rainfall rate-duration-frequency curve.
c	Constant used in rainfall rate-duration-frequence curve.
c	Izzard's Roughness Factor.
C	Coefficient used in "Rational Method."
D & D_e	Detention on ground surface in inches depth or cubic feet per foot width.
f	Capacity of surface infiltration in inches per hour.
f_o & f_c	Constants used in infiltration capacity curve.
F	Accumulated mass infiltration in inches.
$F(q,i,\Delta t) = F$	Functional relationship of q with i and Δt as parameters used in overland flow routing.
$F_g(Q,I) = F_g$	Functional relationship of Q and I, used in gutter routing.
h	Depth of gutter flow in feet.
i	Rate of rainfall in inches per hour.
i_{ave}	Average rate of rainfall for a given duration, in inches per hour.
I_o	Total instantaneous inflow into gutter in cubic feet per second.
I	Total instantaneous inflow into lateral sewer in cubic feet per second.
I_m	Total instantaneous inflow into main sewer in c.f.s. per acre.
k	Constant used in infiltration capacity curve.
K	Coefficient of overland flow containing Izzard's "c."
ℓ	Length of overland flow strips in feet.

L_g Length of gutter in feet.

n Manning's coefficient of roughness.

P Accumulated mass of rainfall in inches.

q & q_e Overland flow runoff in inches per hour.

Q Rate of runoff in cubic feet per second or c.f.s./acre.

r Ratio of advanceness of storm pattern.

S_d Overall average depression storage in inches.

S_o Slope of overland flow strips.

S & S_e Volume of storage in gutter in cubic feet.

S_g Longitudinal slope of gutter.

t_f Time measured from beginning of the infiltration capacity curve in minutes.

t_o Time measured from beginning of rainfall in minutes.

t_r Time of travel in sewers in minutes.

σ Overland flow supply in inches per hour.

V Velocity of flow in feet per second.

w Variable width of gutter in feet.

y Water depth at gutter curb in feet.

z Ratio of total width of water surface to the depth at the flow line.

ACKNOWLEDGMENT

The writers wish to express their gratitude to Mr. Henry Hsien Chu, who has helped in much of the development of the "Chicago Hydrograph Method," and to Mr. Nello DiGiusto and Mr. Ganji G. Tanaka, who before the use of the electronic computing machine did much of the laborious calculations. The programming for the computing machine was prepared by Mr. Aizad N. Khan.

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SEWAGE TREATMENT BY RAW SEWAGE STABILIZATION PONDS[a]

Closure by W. W. Towne and W. H. Davis

W. W. TOWNE,[1] M. ASCE and W. H. DAVIS.[2]—The authors are grateful to Mr. Stone for his discussion in which he cites his experiences and observations at the Mojave Test installation at Mojave, California. The experiences here appear to differ from those of the authors, principally in two respects; namely, organic loading and effects of depth on mixing and organic removals.

As stated in the original paper, the relative low loadings as used in the Dakotas resulted in the minimum time of recovery from anaerobic to aerobic conditions during the critical period: i.e., transition from ice cover to open water. Undoubtedly, the accumulation of organic sludge during the winter operation followed by more rapid anaerobic decomposition accompanying spring temperature increases and before an aerobic environment is reestablished, is a major odor potential in pond operation under those climatic conditions. The original paper predicted, however, that much heavier loadings might be handled in the absence of ice cover, although there were no extremely high loadings in the Dakotas at that time.

Mr. Stone's experience at Mojave is a valuable contribution to our knowledge on this question. More recent Public Health Service studies have also confirmed the feasibility of much heavier loadings in the warmer climates.

In some recent observations on ponds with loadings of 200 to 300 lbs. BOD per acre per day the authors have observed algae growths with accompanying dissolved oxygen in the surface layers but with anaerobic conditions prevailing at the lower depths. Perhaps this situation existed at the Mojave installation when the heavier loadings were "accompanied by slight odor."

The question of whether or not to provide some form of conventional sewage treatment ahead of stabilization ponds in order to reduce the winter sludge accumulation and resulting undesirable conditions appears to be largely one of economics. In the Dakotas, the cost of additional land required for the lower loadings may be less than for providing pretreatment.

With respect to the effects of depth on pond operation, here the conditions experienced at Mojave have not been observed by the authors either in the initial Dakota studies or in subsequent investigations and observations. Our experience has indicated that wind induced currents are important factors in

a. Proc. Paper 1337, August, 1957, by W. W. Towne and W. H. Davis.

1. Chf., Water Pollution Control, Robert A. Taft San. Eng. Center, U. S. Public Health Service, Cincinnati, Ohio.
2. San. Engr., Water Pollution Control, Robert A. Taft San. Eng. Center, U. S. Public Health Service, Cincinnati, Ohio.

dispersing both dissolved and settleable solids throughout the pond, and that the scouring action necessary to prevent accumulation of sludge banks is even more pronounced in shallow ponds than in deeper ones. It is true that high dikes, when ponds are operated at shallow depths, may shelter the entire surface of small ponds and greatly reduce wind-induced mixing. Under such conditions a greater depth might well result in better mixing due to the fact that the liquid surface would be closer to the dike top and less protected from the wind. The authors have not observed short circuiting or bypassing of raw sewage in shallow ponds as reported by Mr. Stone.

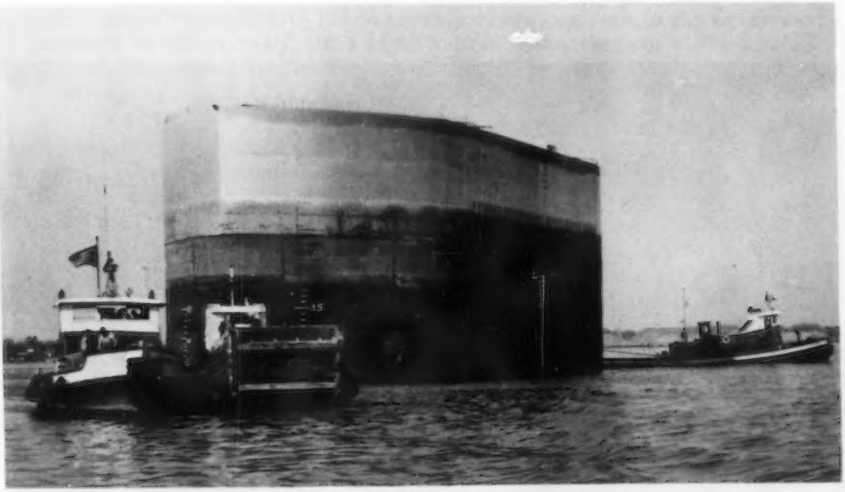
Mr. Stone's discussion adds additional information to the writers' rather limited knowledge of the many factors that may play a part in the application of various theories applied to full scale operating problems. As knowledge in this field increases, there will no doubt be rational explanations for many of these apparent differences in pond behavior.

WATER INTAKES IN THE DETROIT RIVER[a]

Closure by Eugene A. Hardin

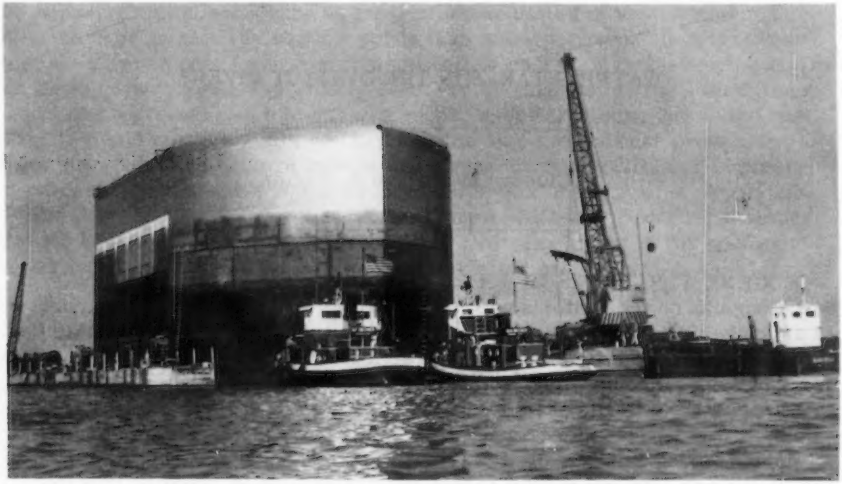
EUGENE A. HARDIN,[1] M. ASCE.—The discussions of Mr. Papp and Mr. Coughlan on the writer's paper contribute a lot to the subject, particularly as to the details of the preliminary studies which were made to locate the best quality of water in which it was practicable to construct the intake. This contribution is sincerely appreciated.

There is little to add at this time except to report that the steel caisson was built during the summer and fall of 1958, and towed to the site and sunk into place during October and November 1958. The photographs, which follow, give an idea of the size of this undertaking. It is expected that the concrete work to be performed within the caisson will be completed during the summer of 1959.

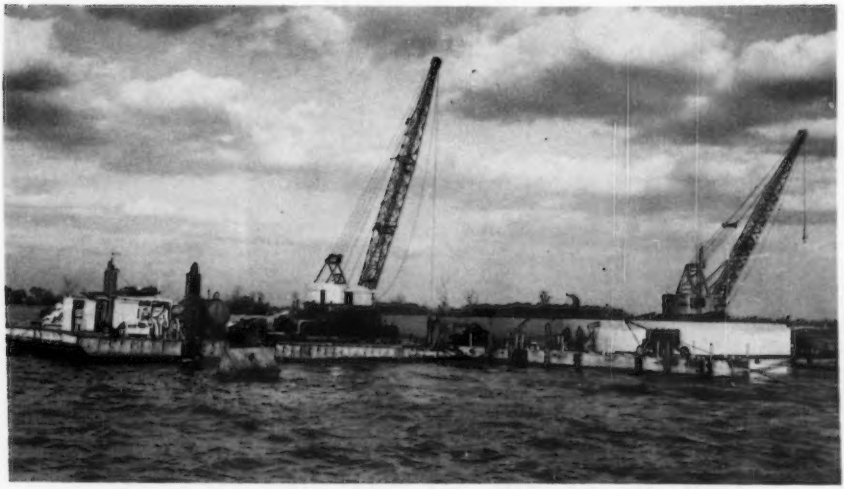


TOWING INTAKE STRUCTURE TO SITE
WAYNE COUNTY, MICHIGAN

a. Proc. Paper 1592, April, 1958, by Eugene A. Hardin.
1. Associate Civ. and San. Engr., Parsons, Brinckerhoff, Hall & Macdonald, Cons. Engrs., New York, N. Y.



INTAKE STRUCTURE AT SITE
WAYNE COUNTY, MICHIGAN



INTAKE STRUCTURE SUNK IN PLACE
WAYNE COUNTY, MICHIGAN

PROPOSED CHANGES IN EASTERN WATER USE POLICIES[a]

Discussion by Howard M. Turner

HOWARD M. TURNER,[1] M. ASCE.—This paper considers the changing use of water in the East and the possibility of using a Model Water Use Act. The author describes the proposed act prepared by the Michigan Law School and discusses it as follows.

1. The proposed act assumes that the riparian or responsible use doctrine now used in the eastern states should be changed. This is a very great change indeed. The present water rights go back for a great many years to the time of the old Mill Acts which give a man a right to erect a dam and to flow land above it provided he does not interfere with anybody's mill rights and pays damage for the flowage excess. The present doctrine of responsible use has been well described as follows:

> "A man who owns land through which a stream of water flows has a right, which is incident to his ownership of the land, not to the water as property in and of itself, but a right to have that stream of water flow as it would flow under natural conditions, and to make such use of it while it passes through his land as it is possible and practicable to make without interfering with a similar right that is possessed by his next neighbors below him . . . It does not mean very much to say that a man is entitled to have a natural stream flow through his land as it naturally flows, because under present conditions of population and civilization there are many things which interfere, and have interfered, with the natural flow of streams, and which make their flow, as they exist today, different from what it was when the country was in a state of nature, as it were . . . But the meaning of that principle of law is that, taking the thing by and large, an owner of land through which a stream flows is entitled, as nearly as it is possible to obtain it, to the uninterrupted ordinary flow of the stream under the conditions which have naturally grown up along the borders of that stream."[2]

This right has been so firmly established that in the old, large water powers in Maine and Massachusetts, the right to use a certain flow of water was made a part of the land and was deeded with it by indenture forever. The difficulty of changing such established rights which have been successfully in use for over one hundred years would be extremely great.

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a. Proc. Paper 1777, September, 1958, by Murray Stein.

1. Cons. Engr., Boston, Mass.

2. Journal of the New England Water Works Association, "The Underlying Principles Governing Riparian Water Rights and Diversion Suits," Charles F. Choate, Jr., Vol. 35, No. 3 September, 1921.

2. The author states that the proposed act "may lack sufficiently precise legislative standards and criteria." As it is drawn, the commission which is set up will determine what is the "best" beneficial use of the water. This puts the decision up to the views of the members of the commission. To draft detailed laws to cover adequately all the various and possibly changing conditions existing on all our rivers would be very difficult indeed when all the different circumstances of each are considered.

3. The author states that there "may be" doubt as to the constitutionality of taking water from a riparian owner. It seems to the writer that this is an understatement and that the taking of rights definitely established as appurtenant to the land from early days is certain to be challenged. It seems clear that if such rights were taken, the owner would have to be compensated. This would be extremely difficult and expensive work.

4. The author lists water uses under seven headings; the last one combines "power development and the transportation of wastes." The writer does not know whether this follows the Model Act, but the uses of water for power and for the transportation of wastes are entirely different, the former involving changes in natural flow. The principles involved are so different that these uses should be kept separate.

5. The author pertinently points out the difficulty of giving special consideration for domestic use, considering that all water supplies furnish many other varied uses in addition to domestic supply.

6. The author points out that the characteristics of the individual drainage basins are so unique that the development of their resources might better be considered on an individual basis. It seems to the writer that this is one of the advantages of having these matters regulated by the broad principles of common law and ownership with the minimum amount of statutory regulation possible so that the different characteristics of the nature and use of the different drainage basins can be considered without trying to specify them in detail for all.

7. It is necessary to have a certain amount of regulation which will tend to increase as the uses of water increase in quantity and diversity. It seems to the writer that the more this can be confined to as little as possible specific regulation on different basins and in different states under the present all-inclusive riparian rights now in existence, the better the modern requirements can be established. The author states that to change the law "may very well disrupt smooth and established operating programs with the states." The writer is sure it is certain to do so.

The author states that the East has been presented with two choices: "(1) a revision of existing law into a single administrative code to be administered by a single State water resources agency; or (2) the utilization of present water law (with such modifications as may prove to be needed) administered by the present existing operating agencies in cooperation with a State-wide coordination, planning and development agency." The writer would express this second alternative as follows: the utilization of the water resources under the present basic riparian reasonable use rights restricted only where necessary to cover certain specific, local needs and to furnish for the use of interstate water cooperation as may be advisable with other states.

RESISTANCE OF SEWAGE SLUDGE TO FLOW IN PIPES[a]

Discussions by M. B. McPherson and Harold E. Babbitt

M. B. McPHERSON,[1] A. M. ASCE.—Mr. Chou's attempt to correlate published information is commendable. However, several serious limiting conditions have not been mentioned and oversimplification in some instances has led to erroneous conclusions.

Babbitt and Caldwell Studies

In reference (1) an equation for steady, laminar, plastic flow was developed, starting with the premise that "the sludge flowing in the center of the pipe moves as a solid plug." It was then reasoned that the shear stress at the outer radius of the "solid plug" must be equal to the yield value. Assuming that pressure is hydrostatically distributed in a vertical plane of the flow section, or $\partial_p/\partial\ell$ is a constant, a linear variation in shear stress from the outer edge of the "plug" must be constant and equal to the yield value (it was erroneously shown decreasing linearly to zero at the center of the pipe on p. 13 of the reference, but the derivation leading to Mr. Chou's Eq. (10) did not involve this error and is correct).

The principal objective of the Illinois studies was to validate the functions in the derived equation(s). Several types of apparatus and methods of measurement were employed. Correlation between the different types of measurement in terms of yield value and coefficient of rigidity was incompletely stated and inconclusive. Mr. Chou has included data acquired from different types of measurement which do not have absolute calibration characteristics in common. Test data which he plotted in his Figs. 2 and 3 include observations from the following installations: a University of Illinois 3/8-inch "long tube (3-ft.) viscosimeter," field tests on full-sized pipe (2", 5", 8" and 12", plus four unidentified values from Great Britain for 4", 9", 10" and 12" pipes and twelve tests on a 7.9" pipe from Stuttgart, Germany) and a special University of Illinois recirculating apparatus (1", 2" and 3" pipe).

Very little of the original data is tabulated in reference (1). In Fig. D-1 is plotted one of three samples of field data given (taken originally from "unpublished data of Chicago Sanitary District"). This sample serves to illustrate the graphical procedure employed by Babbitt and Caldwell to obtain the yield stress and coefficient of rigidity. Eq. (10) is an abbreviation of the basic derived equation (from which a higher order term has been dropped) and is not valid where Sp approaches Sy. The yield value of 0.060 and rigidity

a. Proc. Paper 1780, September, 1958, by Tsung-Lien Chou.
1. Research Engr., Philadelphia Water Dept., Philadelphia, Pa.

FIGURE D-1
(DISC. OF 1780)

SAMPLE OF BABBITT AND CALDWELL CALCULATION PROCEDURE

coefficient of 0.021 are specific to the conditions of this particular test. The only comparative data given for the different testing methods are as follows:

Decatur, Ill., Imhoff:

Method of Measurement	M	Sy	η	°F.
3/8" long-tube viscosimeter -	88.3	0.405	0.035	61°
Recirculating device (2"?):				
"Before agitation" -	86	0.12	0.017	60°(?)
"After agitation," (3-hr. recirculation) -	86	0.065	0.0165	60°

Decatur, Ill., Digested:

Method of Measurement	M	Sy	η	°F.
3/8" long-tube viscosimeter -	86	0.581	0.040	60°
Field tests, 2" pipe -	87	0.141	(0.015)	70°

The writer noted further that non-recirculated sludge physical properties were a function of the test pipe diameter as well as the moisture content. (Babbitt and Caldwell had found very little difference between yield and rigidity values for Decatur Imhoff sludges for 1", 2" and 3" pipes, when recirculated. Apparently they did not recognize the trends of the non-recirculated field data in terms of diameter). Arbitrarily dividing the 3/8" tube viscosimeter (non-recirculated) yield and rigidity values by four appears to take a reasonably consistent account of both the differences in apparatus and in pipe diameter. The results appear in Fig. D-2. There appears to be little probable difference in coef. of rigidity between the Imhoff and digested sludges. The yield stress for a digested sludge appears to be greater than for an Imhoff sludge, but in the 5", 8" and 12" pipe size range, the yield value would probably differ little from that for an Imhoff sludge. The scatter of data may be due to temperature differences, errors in measurement or other factors. Unfortunately, only a range of temperature is given for the field data. The following comprise the sole comparative temperature data given, but since the 3/8" tube was used in the measurements, transferability to full-sized pipes is highly questionable.

Effect of Temp.; M = 93%; 3/8"D; Danville, Ill., Digested:

Ref. Test No.	Temp., °F.	Yield Value as per cent of yield value at 40° F.	Coef. of Rigidity as per cent of coef. of rigidity at 40° F.
14	46°	114%	112%
16	71°	88%	73%
15	122°	35%	54%

The data from Stuttgart divide into two unexplained separate and distinct trends and differ considerably from the American data values. Inasmuch as Prof. Babbitt was intimately acquainted with the Calumet, Illinois treatment plant and field data, but not the German, there appears to be no practical basis for extended speculation as to the causes of these differences.

Mr. Chou states: "There is no experimental determination of critical Reynolds number for sludges." However, "observed critical velocities" can be converted to Reynolds numbers. The "observed critical velocity" was

FIGURE D-2
(DISC. OF 1780).

SELECTED AND REVISED REF. (1) (TABLE 8) DATA
S_Y AND η

taken by Babbitt and Caldwell as the least velocity where the steep head-loss characteristic of turbulent flow first appeared to be established. As may be seen in Fig. D-1, there is a transition between Eq. (10) and the critical point. The only critical velocity values cited for full-sized pipes are for 8" and 12" diameters for Calumet Imhoff sludges (test numbers 26 to 40, and number 90, for M = 88-94%). Using Eq. (3) the writer computed R for these tests and found a range of 1,800 to 3,200, with a mean for the 16 tests of 2,400. Therefore, Eq. (6) for 2,000 and Eq. (7) for 3,000 must be regarded as theoretical and their literal usage, as in Table 4, is unwarranted. If turbulent flow is to be avoided, it would appear that a limiting value of about 2,000 would be a reasonable figure to use. The author's use of three and four significant figures in his several calculations might easily mislead a reader as to their true accuracy. It should be noted in passing that the critical velocity for sludges is of the order of 10 to 100-times that for water.

In reference (2) the statement partially quoted by the author: "In order that such formulas may be applied (such as Hazen and Williams), the sludge must be made up of finely-divided substance dispersed in water, and the velocity of flow must be above the critical," referred to their tests on six clay slurries and a digested sludge in 1/2", 1", 2" and 3" pipes. The single sewage sludge is not described and is unsupported by data or plots of any kind. All seven turbulent flow tests were run using a recirculating device. It would be expected that the recirculated sewage sludge exhibited higher turbulent flow head losses than water, as was the case in reference (1), Fig. 20, with Decatur, Illinois Imhoff sludge recirculated through 1", 2" and 3" pipes. The statement quoted above may have been intended to apply only to slurries, or to slurries and recirculated sewage sludges. In comparison, in Table 6, for Calumet Imhoff tests with an 8" pipe (not recirculated), it is shown that a Hazen-Williams C of 121 for water flow matched the observed turbulent flow velocities and heads for the sludge. In Fig. D-1 (sludge not recirculated) the critical velocity point is satisfied with a C of 112 for the 5-inch pipe. Since both of these instances indicate a C-value as high or higher than one might expect for these sizes for water, it would appear that the head loss for turbulent sludge flow without recirculation might not be significantly greater than the corresponding head loss for water. To repeat, after a long period of recirculation, Imhoff sludge head losses were significantly higher than those for water (reference (1), p. 48, Fig. 20). It is quite possible that the turbulent flow characteristics of recirculated versus comparatively undisturbed sludge are influenced by inherently different degrees of thixotropy.

Greenwich Experiments by Brisbin—Ref. (3)

It is to be particularly noted that all data given by Babbitt and Caldwell were for steady flow. The raw sludge flow in Brisbin's tests was driven by one of two reciprocating plunger pumps. For measurements of head loss "the gauges were read at the maximum pressure for each stroke." Brisbin observed that "The head losses per length of pipe decreased as the distance from the pump increased," which would be expected with unsteady, cyclic flow, particularly with a highly viscous fluid. The distances from the pump to the test sections are not described; different positions of test section relative to their distance from the pump would be related to different local peak ("surge") velocities and local peak pressure hydraulic gradients. The velocities given are those for the "surge flow rates." The "surge velocity" for this

type of plunger pump is probably between 300 and 400% of the average transporting velocity. If the surge velocity was transmitted undiminished and instantaneously through the entire 60-ft. length of the welded steel test section, one would expect the peak hydraulic gradients (i.e. those measured) to be in the order of 9 to 16 times the gradients corresponding to the average transporting velocity.

From the Brisbin data one can logically conclude only that the results represent a special, incompletely defined installation wherein measurements made under conditions of pulsating flow were restricted to peak pressures. To suggest that the resulting data can be applied to steady flow (or, be compared with the steady flow data presented by Babbitt and Caldwell) is misleading and incorrect. Mr. Chou's premise that the raw sludge (5-9% solids) tests with the O.D.S. pump, 4-in. pipe and surge velocity of 5.1 f.p.s. represented turbulent flow is purely speculative. If turbulent flow occurred in these tests at all, it was a momentary condition and could not have persisted through more than a fraction of a full pump cycle.

Since most sludge pumps are of the reciprocating plunger type, the need for extensive knowledge of unsteady, laminar and turbulent flow characteristics is particularly acute. Little progress in this direction will be achieved until the principal corresponding steady flow characteristics are more firmly established.

General

The units for the coefficients of rigidity and viscosity have been given incorrectly as pounds per foot per second, whereas they are pounds per foot-second; this error occurs also in reference (1) but has been corrected in reference (2). The lower-case v which appears in several of the equations is intended to be the average velocity as represented by the discharge divided by the pipe cross-sectional area. In Eq. (8) the coefficient of viscosity rather than rigidity should appear; to be precise, the equation should be multiplied by 32 divided by the acceleration of gravity.

The writer agrees with Mr. Chou, after studying the clay slurry data of references (1) and (2), that "there is no sound reason to amalgamate sewage sludge, clay slurry, wood pulp and drilling mud in one pack - - - -," at this time. Mr. Chou has rendered the profession a timely service. His review and this discussion should serve to emphasize the need for much more complete and correlated field and laboratory data.

HAROLD E. BABBITT,[1] M. ASCE.—The author states in his synopsis that: ". . . the flow of sludge in pipes defies direct application of existing hydraulic formulas for computing head loss." This is a discouraging statement which the author soon refutes when he says: ". . . formulas are developed for practical application"; and later: ". . . it is believed that a designer can predict the head loss of sewage sludge flowing in pipes under any conditions." With such encouragement the designer may be led to pursue the author's development of how this is to be done.

If it be agreed that sewage sludge may flow either as a liquid or as a plastic the conception of its flow characteristics is easier to comprehend. It

1. Engr., Columbia, Mo.

has been shown that these flow characteristics depend, in part, on the character and concentration of the suspended solids, and on other conditions. A perfect fluid may be considered to be a substance which, if confined in a pipe, will attain velocity of flow upon the application of the slightest pressure to one end of the liquid in the pipe. In the region of straight-line or laminar flow the velocity attained will increase in direct proportion to the magnitude of the force applied. When the magnitude of the applied force becomes sufficient the character of the flow will change from laminar to turbulent, passing through a region of "mixed flow", combining both laminar and turbulent flow. Water and very thin (dilute) sewage sludge are physically so close to a perfect fluid as to display such flow characteristics.

Plastic flow is manifested by various phenomena, one of which is common to all plastics. This is the need of applying a measurable force before a measurable velocity will be created. After a force of sufficient magnitude to create velocity of flow has been reached the application of increased force will increase the velocity, but not necessarily in direct proportion to the increase in the magnitude of the force. In normal sewage sludges, however, the increase in velocity, once velocity is started, is directly proportional to the increase in force applied. For relatively dilute sludges, such as are normally encountered in sewerage practice, the rate of increase in velocity in the laminar region, will be proportional to the rate of increase of force. As in liquid flow the character of flow changes from laminar to turbulent upon the application of sufficient force.

If the relationship between force applied and velocity of flow created be plotted logarithmically, the lines in both the laminar and turbulent regions will be straight. The intercepts and slope of the line for the turbulent region have been determined empirically for water by many investigators and have been incorporated in such well-known hydraulic formulas as Mannings and Hazen and Williams. In the normal application of these formulas to the solution of problems in the flow of water or of sewage no allowance is made for the specific gravity, viscosity, or other characteristics of the liquid. However, if the temperatures involved are above normal allowance is usually made by the determination and application of Reynolds number in the solution.

A limited amount of particulate matter can be suspended in water without affecting the characteristics of its flow as a liquid. It has been assumed that when the concentration of suspended solids is such that the solids are in contact and must slide across each other, plastic flow begins. Suspended solids are usually sufficiently concentrated in sewage sludge to create plastic flow. If lines are plotted logarithmically the straight-line portions in laminar and turbulent flow can be expressed in the form shown in the author's formula (12) but with different coefficients and exponents. Conditions affecting these coefficients and exponents include specific gravity, coefficient of rigidity, coefficient of viscosity, and roughness of pipe walls. As the author has inferred no precise formula can be derived nor devised that is applicable to the flow of all sewage sludges in all conduits. Coefficients and exponents must be determined by test for each sludge and material involved. When such parameters are known, formulas (10) and (11) are applicable within the limits of formulas (6) and (7). Specific gravity, viscosity, and rigidity can be determined by laboratory measurements. Since the characteristics of sewage sludge at any plant may vary appreciably the work involved in measuring these parameters may be great. Where a design is to be made for future conditions the magnitudes of the parameters must be assumed.

The author has suggested a statistical method for circumventing the need to determine the value of each constant for each sludge involved; his method possibly being applicable to the flow of normal sewage sludges. His method should result in a solution somewhat more precise than that given by assuming that all sludges flow like water. The author's method may not be so precise as the use of formulas (10) and (11) but it may be easier to apply to many sludges. The formulas require the determination of S_y; η; ρ; and possibly w for each sludge to be studied. It should be possible, however, to use these formulas by applying parameters that have been found from other sludges within the range of common experience.

The author's point that the application of formula (11) to turbulent flow is questionable due to the possibility of uneven velocity distribution across the cross section of the conduit is well taken. The advisability of the application of this formula to the determination of head losses for velocities much above upper critical is admittedly questionable. This fact was not emphasized by the writers when formulas (10) and (11) were first published.

The author's contribution to knowledge of the conditions involved in the flow of sewage sludge is appreciable. The design of sludge pipes and pumps without consideration of the points the author has made may explain some sludge-pump failures and difficulties, or some installations may be uneconomically over-designed. Problems involved in the flow of sewage sludges exist also in the flow of industrial sludges, slurries, and suspensions. A general solution, applicable to all sludges would be applicable in fields other than in the field of sewage treatment.

PLANNING THE FUTURE FOR CHICAGO'S WATER SYSTEM[a]

Discussion by Herbert Moore

HERBERT MOORE,[1] M. ASCE.—Although Chicago's water supply appears to be at its door step, there are numerous hazards and problems most familiar to waterworks engineers which over the years might tend to threaten continuity of service. A raw water reservoir located above the level of the city for processing Lake Michigan water at uniformly good quality and temperature to meet fluctuating demands, or as an auxiliary source, would appear to be worthy of serious consideration.

The writer made this suggestion ten years ago in a paper presented to the Northern Illinois Waterworks Institute prior to the Navy Pier site expansion program and proposed for Chicago an intake and reservoir north of Racine, Wisconsin. Subsequently a study of the terrain indicates the best location for a reservoir is about ten miles west of Lake Michigan at a location westerly from the cities of Racine and Kenosha. Illustrated on the accompanying map (Fig. 1), there, it would appear, is the largest possible reservoir site which could be developed for processing Lake Michigan water for Chicago. Roughly parallel to Lake Michigan it is a valley with narrow constrictions at each end where the construction of mile long earth dikes would create an artificial lake at an elevation of 150 feet above Lake Michigan with a maximum depth of fifty feet when pumped full of Lake Michigan water.

It is noted that the Illinois State Legislature in 1957 passed bills which now authorize Chicago to build and operate waterworks facilities as far as 35 miles beyond the city limits. Although a suitable smaller reservoir site might be developed within that limit, it occurs to the writer that the State Legislature might be prevailed upon to amend and increase the arbitrary limit to extend to the Wisconsin State line, and to provide for the execution of such corporate arrangements as might be necessary for construction of waterworks facilities in another state.

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a. Proc. Paper 1782, September, 1958, by W. W. DeBerard.

1. Cons. Engr., Milwaukee, Wis.

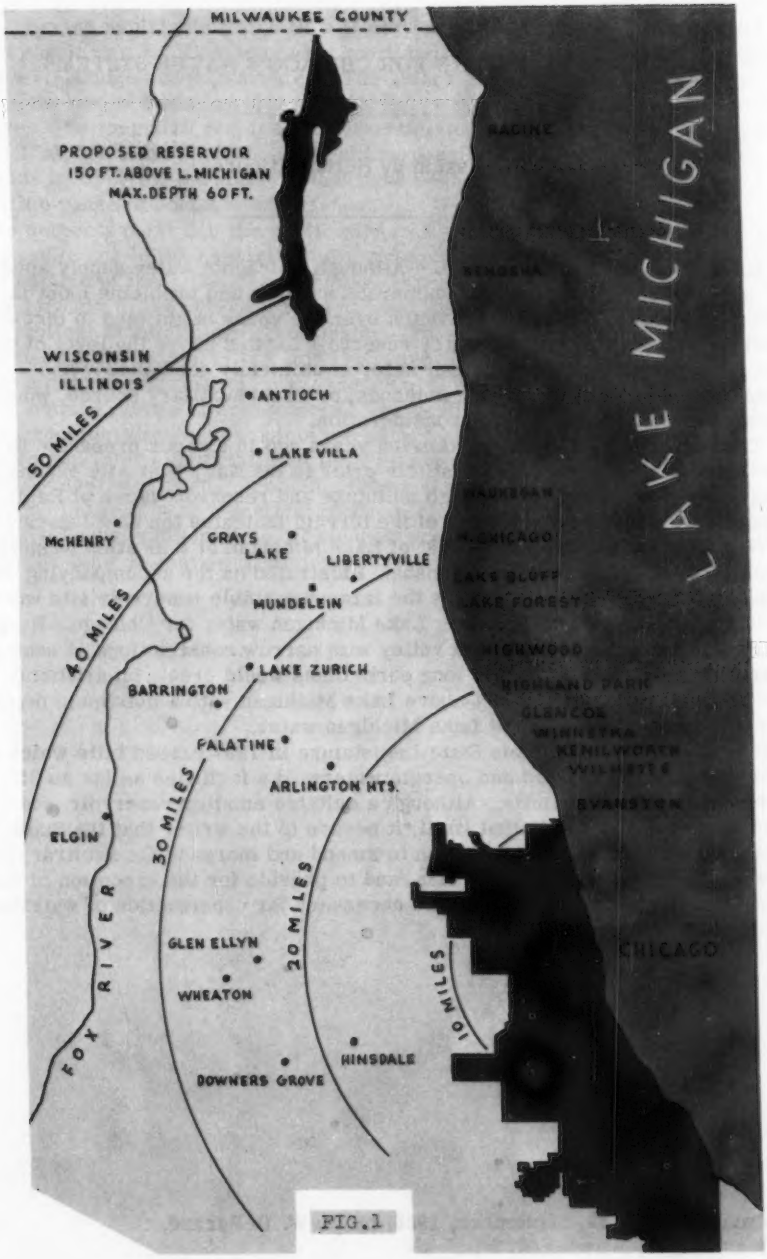
MILWAUKEE COUNTY
PROPOSED RESERVOIR
150 FT. ABOVE L. MICHIGAN
MAX. DEPTH 60 FT.
RACINE
KENOSHA
LAKE MICHIGAN
WISCONSIN
ILLINOIS
50 MILES
ANTIOCH
LAKE VILLA
McHENRY
GRAYS LAKE
LIBERTYVILLE
MUNDELEIN
40 MILES
LAKE ZURICH
BARRINGTON
PALATINE
ARLINGTON HTS.
ELGIN
30 MILES
20 MILES
10 MILES
FOX RIVER
GLEN ELLYN
WHEATON
HINSDALE
DOWNERS GROVE
WAUKEGAN
N. CHICAGO
LAKE BLUFF
LAKE FOREST
HIGHWOOD
HIGHLAND PARK
GLENCOE
WINNETKA
KENILWORTH
WILMETTE
EVANSTON
CHICAGO

FIG. 1

AN ANALOG COMPUTER FOR THE OXYGEN SAG CURVE[a]

Discussion by Russell H. Babcock

RUSSELL H. BABCOCK,[1] A. M. ASCE.—The authors have presented a new approach toward the solution of oxygen sag curves. Obviously, there is an increasing need for computing devices such as they describe, as more and more advancement is made toward solving the problems of stream pollution.

In limiting the paper to the use of the electric analog computer, the authors have overlooked other available computing devices which may have inherent advantages over the analog type for this type problem. Electric analog computers are available for solving problems of the successive approximation type, such as Hardy-Cross pipe networks, but for many other computing problems, they are too complicated and expensive. Moreover, they are normally inflexible . . . customarily built to fit a single equation or solve a particular problem.

Pneumatic computers, though less publicized, have been used successfully for solving problems with established equations. The pneumatic computer has proved to be easier to operate and less expensive than its electric counterpart.

Commercial pneumatic-mechanical computing relays have been available for several years for handling specific mathematical functions. These devices operate in response to a pneumatic signal. The constants and variables, instead of being electrical values as with electric analog computers, are specific air pressures within a 3-15 psi range, and they can be recorded or indicated with relatively inexpensive conventional pressure instruments.

The following is a partial list of the functions that can be performed by combinations of computing relays, pneumatic capacitors and restrictors:

A, B, C, = Variables

K,K' = Constant

$\frac{A + B}{K}$	KA	
$\frac{A - B}{K}$	$\frac{A}{K}$	A^2
$K - A$	AB	$Ae^{-K'/t}$
$A + KB$	$\frac{AB}{C}$	$A\ (1-e^{-K'/t}$
$A - KB$	$\sqrt{A}$	$\frac{dA}{dt}$

a. Proc. Paper 1850, November, 1958, by Morton D. Sinkoff, C. Don Geilker and Jan G. Rennerfelt.

1. Mgr., Water & Waste Div., The Foxboro Company, Foxboro, Mass.

An important, and somewhat difficult, control problem which will be handled by pneumatic computers is the computation for the speed control of the pumps at the Deer Island Pumping Station, presently under construction for the Metropolitan District Commission, Boston, Massachusetts. One of the equations to be solved at this installation is:

$$Q_c = Q_1 + Q_2 + B_1H_1\,(1-e^{-t/T_1}) + B_3H_3e^{-t/T_3}$$

Q_c, Q_1, Q_2 Flows

H_1, H_3 Levels

B_1, B_3, T_1, T_3 Constants

The complexity of this equation results from the raw sewage being carried to the plant through tunnels ranging up to seven miles long. The resulting hydraulic characteristics and lag times necessitates using measurement data at the inlet point of a tunnel to compute the set point for flow from the pumps. A prototype of the pneumatic computer designed for this installation has been successfully operated.

A computer for solving the oxygen sag curve is illustrated in Fig. 1. The variable L_a

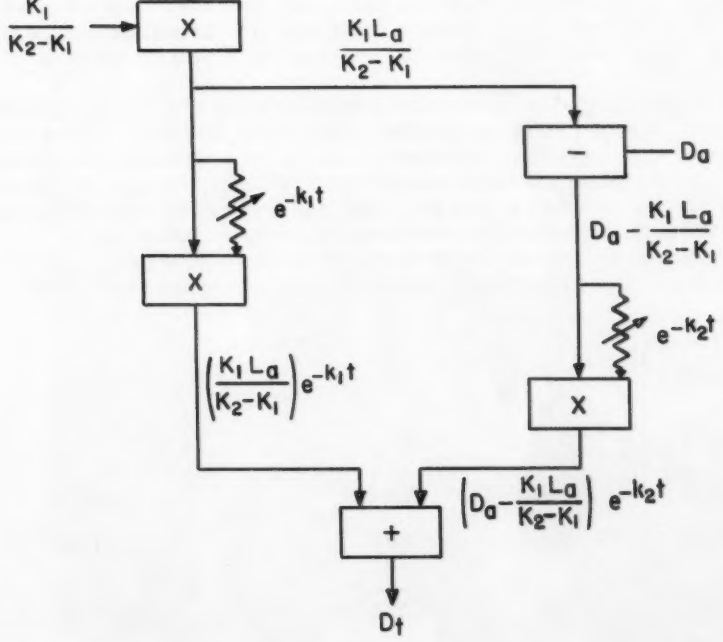

FIG. 1 SCHEMATIC DIAGRAM PNEUMATIC COMPUTER FOR OXYGEN SAG CURVE

is a pneumatic signal fed into the computer circuit. The constants K_1, K_2, and Da are multipliers mechanically fed into the circuit, and the exponential values of e are restrictors between sections of individual computing relays. The resultant value, D_T, is a varying pneumatic signal which is recorded as a trace of the oxygen sag curve. A computer of this design could be readily assembled from standard computing relays manufactured by several industrial instrument companies.

An interesting advantage of this type of computer is that the same computing relays can be arranged in a variety of ways for solving additional equations, merely by using flexible tubing to connect the ports in different sequence. Thus, it is possible to build a universal computer for relatively simple equations.

Pneumatic-mechanical computing relays are common, but often overlooked for computing purposes, because they are not practical for problems requiring successive approximations. They are, however, very practical for solving a known equation requiring a single solution. Proving their value for such purposes as averaging flow signals, subtracting temperatures, biasing control signals, and many others, these computing relays are being used extensively throughout industry.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 (SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 (CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968 (SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977 (WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

c. Discussion of several papers, grouped by divisions.

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MARCH 1959 — 14 NO. SA 2
VOLUME 85 PART 2

Your attention is invited

NEWS OF THE SANITARY ENGINEERING DIVISION OF ASCE

JOURNAL OF THE SANITARY ENGINEERING DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

March, 1959

TENTATIVE PROGRAM
SANITARY ENGINEERING CONFERENCE
JANUARY, 1960
NETHERLAND HILTON HOTEL
CINCINNATI, OHIO

January 6, 1960	A. M.	An outstanding authority will give the purpose of the Conference and outline, from abstracts and papers received from the authors, what may be expected during the three panel discussions.
	P. M.	Panel on Air Pollution
January 7, 1960	A. M.	Panel on Land Pollution
	P. M.	Panel on Water Pollution
	Evening	The Cincinnati Section of the ASCE will provide a Cocktail Hour, followed by a banquet, with light entertainment for the remainder of the evening.
January 8, 1960	A. M.	An outstanding authority will tie together the discussions of the three panels, along with his own comments, to give to all the benefit of the meat of the program.
	P. M.	Inspection trip through the Mill Creek Sewage Works.

CONFERENCE ON ELECTRONIC COMPUTATION PAPERS AVAILABLE

In response to the unprecedented demand expressed by participants in the Conference on Electronic Computation, the twenty-five papers presented at the Kansas City Conference on November 20-21, 1958 are being offered by the Society in a single paper-bound volume. This special edition is composed of the twenty-two technical papers and the welcome address, the keynote

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address, and the luncheon address, all as delivered at the Conference. The prices (post-paid) are as follows:

First Copy	$ 5.00	$10.00
Additional Copies	$10.00	$10.00

Orders should be sent to Society headquarters in New York City.

* * *

SANITARY ENGINEERING EDUCATION

Sanitary Engineering Research Laboratories and Graduate Program at Washington University, St. Louis, Missouri

Washington University in St. Louis, Missouri, has long had an outstanding reputation in the field of medicine, and therefore a strong interest in problems of public health. During these last five years an increasing number of requests for trained sanitary engineers and for specialized investigation of public health problems have alerted the University's administration to their responsibilities for establishing research and training facilities in sanitary engineering to serve this midwestern area. It was their belief that through the support of the University's well known schools of chemistry, biology, botany, and mathematics, a program in sanitary engineering would grow rapidly in the Department of Civil Engineering. Therefore, steps were taken to establish such a program.

Professor Henry M. Reitz had already established comprehensive courses in hydrology and hydraulics. As Acting Head of the Civil Engineering Department, he decided to bring in a Director for an expanded program in sanitary engineering. In 1956 Dr. D. W. Rychman joined the Washington University staff. One year later Professor E. Edgerley, Jr., came in to head the sanitary engineering laboratory courses. At the present time the sanitary engineering staff consists of Dr. N. C. Burbank, Jr., Professor of Sanitary Engineering and Head, Department of Civil Engineering; Professor Henry M. Reitz, Professor of Civil Engineering, specializing in soil mechanics and hydraulics; Dr. D. W. Rychman, Associate Professor of Sanitary Engineering and Director of Sanitary Engineering Laboratories and Graduate Program; Professor E. Edgerley, Jr., Assistant Professor of Sanitary Engineering; Mr. S. Gigoropoulos, Research Instructor of Sanitary Engineering; Miss G. Gross, Sanitary Engineering Laboratory Coordinator; and Mrs. N. Grosswasser, Research Instructor in Sanitary Engineering. The new Sanitary Engineering Research and Graduate Laboratories are located in the modern air conditioned NEW ENGINEERING LABORATORY BUILDING which is adjacent to the existing engineering buildings. Over 3600 square feet of space has been assigned for the Sanitary Engineering Laboratories which include the following:

Sanitary Microbiology Graduate Laboratory
Sanitary Microbiology Research Laboratory
General Sanitary Engineering Graduate Laboratory
Sanitary Engineering Research Laboratory
Sanitary Chemistry Research Laboratory

These laboratories are equipped with modern instruments and apparatus required for research in this field. These facilities include: four walk-in type constant temperature rooms, spectrophotometers, Kjeldahl Nitrogen Apparatus, Warburg Respirometer, Van Slyke Apparatus, settling columns, ovens, furnaces, microscopes, micro-photometric equipment, column chromatographic apparatus, and the other usual sanitary engineering laboratory apparatus.

The degrees of Master of Science and Doctor of Science in Sanitary Engineering are offered. Outstanding students with either civil or chemical engineering undergraduate degrees are presently enrolled in the graduate program. Chemistry, biology, and botany students who wish to specialize in sanitary science are especially welcome. Over 30 semester credit hours of formal classwork is presently offered with opportunities for research, thesis credits, and additional coursework in the Departments of Chemistry, Biology, Botany, Mathematics, and the School of Medicine.

Present active research project interests are as follows:

Characterization of Organics of Taste and Odor Significance from Natural Waters
The Biochemical Degradation of Various Industrial Wastes
The Investigation of Selective Oxidation of Certain Organics with Chlorine Dioxide
The Ozonation of Organics of Taste and Odor Significance
Bioassays of Industrial Waste
Chemical, Physical and Biological Treatment of Specific Industrial Wastes
The Chemical Behavior of Ammonia, Organic Nitrogen and Phenol with Chlorine and Water at High pH Values
The Utilization of Aromatic Organics with Activated Sludge

Other details of the program, including available scholarships, fellowships and research assistantships, may be obtained by contacting Dr. D. W. Rychman, Director of Sanitary Engineering Research Laboratories and Graduate Program, Washington University, St. Louis, Missouri.

Robert A. Taft Sanitary Engineering Center

The Training Program of the Sanitary Engineering Center has conducted a total of seventeen residence courses for 332 trainees in the fields of water supply and water pollution control, milk and food sanitation, radiological health, and air pollution during the last six months. These courses are the traditional short-term technical courses given at the post-graduate level which reflect the latest technical developments and research in the sanitary engineering field. A total of seven field courses were given to 219 trainees during the same period.

The Air Pollution Training Unit of the Center Training Program presented for the first time in January, a course entitled "Chemical Microscopy for Analysis of Air Pollutants," which was attended by twenty trainees. The purpose of the course was to teach the use of chemical microscopy as a tool in the identification of air-borne particulates. Dr. Walter C. McCrone, President, Walter C. McCrone Associates, Chicago, assisted in the preparation and conduct of the course. Dr. McCrone is recognized as one of the Nation's leading authorities in this field.

A new two-week course, "Medical and Biological Aspects of Radiological Health," was given in October. This was attended by thirteen trainees including seven physicians. A principal objective was to stress the hazards of exposure to low-level radiation sources which are becoming of increasing interest to public health personnel.

* * *

SANITARY ENGINEERING RESEARCH

Earle B. Phelps Sanitary Engineering Research Laboratory

The new sanitary engineering research laboratory at the University of Florida has been designated the "Earle B. Phelps Sanitary Engineering Research Laboratory," in recognition of the distinguished service which the late Professor Phelps rendered to the sanitary engineering profession and to the University of Florida.

Professor Phelps came to the University of Florida in 1943 as a research engineer and was an active worker there until his death in 1953. He has been credited with primary responsibility for much of the progress made in sanitary engineering teaching and research at the University. He was retired as Professor Emeritus from Columbia University, and long before he came to Florida he was popularly known as the "Dean of American Public Health Engineers."

Just two weeks before his death, he was selected by the American Public Health Association for one of the coveted Lasker Awards. The award was presented posthumously with the following citation:

> "Earle Bernard Phelps (1876-1953), born in Galesburg, Ill., professionally trained at the Massachusetts Institute of Technology, a dedicated student of William T. Sedgwick, gave a liftime in exploring, discovering, teaching, and applying natural sciences to save mankind from the hazards of biological, physical, and chemical factors of environment. A gifted teacher, he shared generously his wealth of knowledge and experience with a multitude of associates and students.
>
> "He was bacteriologist with the Massachusetts State Department of Health at the Lawrence Experiment Station for four years. He was for fifteen years in the Sanitary Research Laboratory at his Alma Mater, the last five of these as Associate Professor. He was for five years with the U. S. Geologic Survey, and at the Hygienic Laboratory of the U. S. Public Health Service for six years.
>
> "Then he became Professor of Sanitary Science at Columbia University for eighteen years until his retirement in 1943, after which he was Research Engineer at the University of Florida in Gainesville until his death. His major contributions to the science and practice of public health were in water chlorination, stream pollution indices, sewage treatment and disposal, aerial sanitation, milk pasteurization and reconstruction, and shellfish control.
>
> "His Public Health Engineering volume is an educational classic. Charming modesty marked Earle Phelp's life of distinction."

Man and His Environment in Space

A symposium on "Man and His Environment in Space" held at the recent national meeting of the American Association for the Advancement of Science in Washington, D. C., emphasized the immensity of environmental problems associated with long space flights. The cycling of food, water, and air was reported to be a difficult, "bitter," problem. The gradual build-up of toxic substances introduced in minute quantities during each cycle of use was cited as one of the major difficulties. Another major problem is that of making the equipment small enough and light enough to carry in a space vehicle. Usable cycling devices were said to be years in the future.

In the section of his paper on the cycling of air, Bruce W. Pinc of the Air Force Ballistic Missile Division, reported that a system using algae to convert the carbon dioxide exhaled by humans back into oxygen would be so large in the present state of the art, it wouldn't be worth using for flights under 150 man-days. He said it would be more practical to carry oxygen.

William T. Ingram, Professor of Sanitary Engineering at New York University, said that water recycling systems now possible would produce water unfit to drink on a continuing basis after the water had been recycled only a few times. There are a great variety of toxic substances such as phenols, skatole, and indole, that will build up in the water supply and soon make it poisonous unless steps are taken to remove each one. This, he indicated, may make the whole idea impractical. Dr. Ingram said that the use of algae for the treatment of sewage is completely unsatisfactory and that more research and entirely new concepts would be required.

Of the seven methods of water purification being discussed for space vehicles, the use of the sun for distillation and separation by freezing seem to look the best, he stated.

Air Pollution Research Contracts Negotiated

Under the authority provided by Public Law 159—which provides for research and technical assistance relative to air pollution control—the Public Health Service recently negotiated two research contracts.

The first, to be performed by the Southern Research Institute, Birmingham, Alabama, is for development and delivery of a portable, hand-held particle counting photometer, with a particle size analyzer as an accessory attachment.

The particle counting photometer is to be capable of counting particles in the size range of approximately 0.3 to 10 microns diameter equivalent polystyrene latex spheres. The count is to be displayed on an indicating meter and provisions made to permit the attachment of a recorder. A discriminator attachment will permit the lower size cut-off point to be varied over the specified size range. The photometer and its output signal are to be able to function in conjunction with a particle size analyzer as an accessory attachment. The particle size analyzer is to be designed to have a 12 channel output and to be compact, lightweight, and require as little power as possible so that it will be portable and capable of being hand-carried along with the photometer. The simultaneous and cumulative output of each of the 12 channels is to be displayed in recorded form utilizing current photographic or electronic technique. The analyzer should be capable of giving the size distribution of particulates in the size range counted by the photometer.

These instruments are expected to be developed by July 15, 1959.

The second contract is to be performed by the Los Angeles County Air Pollution Control District. It provides for the development of a usable, technically accurate and comprehensive manual of test methods pertinent to the conduct of air pollution source testing and air monitoring activities. The data to be used in the development of the manual will be based primarily on the technical information compiled by and now in possession of the District. The manual material is to be broadly applicable by air pollution control officials, air pollution research and training organizations and designers and operators of air pollution control equipment throughout the country. The manual is to include methods relating to:

1. Testing of air pollution sources.
2. Sampling of the atmosphere for contaminant concentrations.
3. Laboratory analyses of samples.

It is anticipated that this project will be completed by the end of the calendar year.

* * *

WATER SUPPLY AND WATER POLLUTION CONTROL

New Coagulant Aid Approved

The coagulant aid, Ecco Suspension Catalyzer #146, manufactured by the Electric Chemical Company, Cleveland, Ohio, was recently reviewed by the Public Health Service's Technical Advisory Committee on Coagulant Aids for Water Treatment. On the basis of information submitted by the manufacturer, the Committee concluded that this product can be used for treatment of public water supplies with no adverse physiological effect upon water consumers when used in the recommended concentration of 3.5 parts permillion, and provided the quality specifications furnished to the Committee continue to be met. Committee findings bear only on the health aspects of the use of this product and do not constitute endorsement or indicate effectiveness in proposed use.

Orsanco Reports Cleaner Streams

Cleaner streams and the development of robot monitors to maintain a continuous check on their cleanliness are forecast in the annual report of the Ohio River Valley Sanitation Commission. The report was transmitted in December to the governors of the interstate agency created by Illinois, Indiana, Kentucky, New York, Ohio, Pennsylvania, Virginia, and West Virginia.

In presenting the report, Dr. Russell E. Teague, chairman and state health commissioner of Kentucky, said: "When our regional crusade for clean streams was inaugurated ten years ago less than 4 people out of ten in the Ohio Valley were served by sewage-purification facilities. Today the ratio is approaching 8 out of 10. This is but one manifestation of the effectiveness of our interstate pollution control program."

With regard to industrial waste discharges into streams, the report shows that three-quarters of the plants in the Ohio Valley are operating facilities to curb pollution. The number now rated as having adequate control facilities is double that of five years ago.

Word has reached the Division that Ed Cleary, Executive Director and Chief Engineer of the Commission, has assumed a new command—that of Admiral of the Ohio River. This appointment came from Governor Chandler of Kentucky. The Kentucky Admiralty is the maritime equivalent of the Kentucky Colonels.

Low Flow Augmentation for Water Pollution Control

A recent report by C. H. Hull, research associate, Johns Hopkins University, describes an investigation currently underway at that institution for the evaluation of low-flow releases from multiple-purpose water impoundments for water pollution control. This will include the technical, economic, and legal aspects of this potential water use.

Technical features of low-flow augmentation are extremely complex. A determination of the magnitude of regulated flows required to provide a given amount of pollution abatement involves the use of theory and procedures which at best are somewhat controversial. Information on the effects of impoundment on the waste-assimilative capacity of a stream is meager. The theory developed more than 30 years ago for describing the relationships between the supply and demand of oxygen in flowing streams is inadequate to formulate the oxygen balance in impoundments. The effect of wide variations in river flow below hydroelectric power dams on the over-all pollution-assimilative capacity has not been described quantitatively. Mr. Hull, in his review of the problem, asks: "What pollution abatement benefits, if any, are incidental to flow regulation primarily for power production? What compensation flows should be provided below hydroelectric or other storage projects? How low can flows be reduced below hydroelectric dams during off-peak periods without damaging a downstream pollution abatement program?"

One problem related to water storage projects which is becoming widely recognized is the depletion of dissolved oxygen from the deep water of stratified reservoirs. Increasing the flow is only one of two methods of restoring the waste-assimilative capacity lost because of oxygen reduction in impoundments. Natural and artificial reaeration may restore the oxygen content of the discharge. Several methods for artificial reaeration are under investigation.

Many legal and economic problems are involved in low flow augmentation for water pollution control. Investigation of many of these, as well as the technical problems, are underway at Johns Hopkins University. Detailed reports will be issued by that institution as information of value is developed on any single phase of the study.

Zimmerman Process Studies Completed at Chicago

Most sanitary engineers are familiar with the principles of the Zimmerman process for the destruction of organic industrial wastes and sewage sludge. This process operates on the basic principle that organic matter contained in an aqueous solution can be oxidized at high temperatures and pressures and the heat value it contains released for use more effectively than if the water were first evaporated and the residue used as fuel in a conventional boiler.

Under arrangements with the Metropolitan Sanitary District of Greater Chicago, Sterling Drug, Inc., built, equipped, and operated a pilot plant at the

West-Southwest Treatment Works. The District provided the facilities required to deliver to the Zimmermann Process activated and primary sludges, as well as mixtures of both. The plant was completed and ready for operation on September 13, 1957.

A sewage sludge capable of supplying approximately 2350 to 3000 BTU per gallon of liquid sludge was used in these pilot plant studies. Activated sludge at the West-Southwest Treatment Works has a heat value of about 6500 BTU per pound; primary solids, about 7500 BTU. To be thermally sufficient, the sludge must have a solids concentration of 3 per cent; to be self sustaining, 5 per cent solids are needed. Any concentration above 5 per cent will result in a dividend of useful, recoverable energy.

Altogether, 19 runs varying in duration from 26 hours to 361 hours were made over the past year. The temperature of these runs ranged from 488° F to 521° F and pressures were held pretty close to 1200 psi.

The parameters that best indicate the effectiveness of the wet-oxidation system are the COD, which measures the degree of oxidation accomplished, and the reduction in volatile solids. Data show that within the range of solids concentrations studied and under the conditions of operation of the pilot plant (1200 psi pressure and 500° F) the amount of sludge substance oxidized remains fairly constant at approximately 80 per cent and the amount of volatile matter destroyed is fairly constant at about 87 per cent. This relation held true even in the case of the run with centrifuged activated sludge which had an ash content of 45.3 per cent. The data confirm the observation by Mr. Zimmermann that COD removal and its corollary volatile solids removal is a function of the temperature. Percentage-wise, the removal remained constant regardless of the initial sludge concentration within the ranges studied. The 80 per cent reduction obtained in these tests would be improved at higher temperatures and pressures. The reduction of total solids was 66 per cent. Reduction of total solids by anaerobic digestion processes is between 40 and 50 per cent.

The effluent is characterized by high ammonia, high volatile acid, and high BOD content; approximately in the same order of concentration as that associated with the supernatant from anaerobic digestion processes. It is discharged from the reactor as a sterile liquid. After a short period of acclimitization, the effluent discharged is of a quality easily handled by activated sludge treatment. Laboratory trickling filter tests gave similar results. There are no problems indicated in treatment of the effluent.

The range in volatile content of the ash was from 13 to 17 per cent with an average of 15. The ash is biologically stable.

Exhaust gases consist mainly of carbon dioxide and nitrogen, with trace amounts of hydrocarbons. They carry no fly ash and exhibit only a slight odor of pyridine.

It is reported that the Sanitary District is well satisfied with the results of these pilot plant studies and are convinced of the efficacy of the Process. Multiple units, probably four, are now being designed that will treat a total of two-hundred tons of activated sludge per day.

Any inquiries regarding the Zimmermann Process should be directed to Sterling Drug, Inc., 31st Floor, 1450 Broadway, New York 18, N. Y.

Operation and Maintenance Cost Study of Sewage Treatment Works

The Water Supply and Water Pollution Control Program of the Public Health Service has undertaken a study of municipal sewage treatment plant

operation and maintenance costs. This study is expected to contribute useful information for sewage treatment planning at all levels of government.

Cost data will be requested from a selected number of satisfactorily operated sewage treatment plants throughout the United States. These communities will be chosen by state water pollution control agencies on the basis of type of treatment provided, years in operation, quality of operation and maintenance, and availability of operating records.

Survey questionnaires will request information relating to the type of plant, plant loading and performance, operation and maintenance costs, personnel requirements, and laboratory control. Preliminary data obtained during a field trial of the questionnaires in North Carolina exhibited a high degree of correlation between annual operation and maintenance costs and population served, flow, and population equivalent removed by treatment.

Sewage Treatment Works Construction

Communities in 48 states and the Territories of Hawaii and Puerto Rico have participated in the federal construction grants program aimed at accelerating sewage treatment plant construction. Approved projects totaled 1,337 at the end of 1958, with 578 of these under construction and 356 completed.

Two-thirds of all communities receiving construction grants have populations of less than 5,000. Almost 90 per cent will serve communities with populations of less than 25,000. While the communities participating in the program are predominately small, they represent a typical cross-section of municipalities throughout the United States. For example, 3/4ths of the Nation's sewered communities have populations less than 5,000 and 95 per cent have populations less than 25,000.

Construction grants made to communities through 1958 total $113.7 million. An additional $481 million of local funds were raised to construct needed sewage treatment facilities costing an estimated $595 million. Every dollar of federal aid has been matched by more than 4 local dollars.

Grant applications currently being processed or reported to be in preparation will request an additional $96 million of federal aid towards further pollution control construction costing approximately $631 million. Total applications approved, in process, or under preparation, request grant funds 50 per cent in excess of existing appropriations.

Sewage treatment works construction has increased steadily during the course of the construction grants program. Contract awards in 1957 rose to $351 million and it is estimated that the total for 1958 will reach $400 million. The 1952-1956 annual average was $222 million.

In spite of these significant gains, however, much higher annual construction levels must be attained if water pollution from municipal wastes is to be brought under control. Public Health Service estimates indicate that sewage treatment works construction must be maintained at an annual level of $575 million for at least eight years if our current construction backlog, obsolescence and populations growth needs are to be met.

State Construction Grants Endorsed in New Hampshire

The Interim Legislative Council approved a proposal for State aid to municipalities in the construction of sewage treatment facilities at its final policy meeting on October 14. The proposal endorsed by the Council provides

for 20 per cent State grants to supplement the 30 per cent federal grants now available under Public Law 660, and would cost the State about $330,000 annually.

The New Hampshire Municipal Association will sponsor the State-aid plan in the 1959 Legislature and Governor-elect Wesley Powell has indicated his support of the measure. State grant legislation for pollution control facilities was enacted by Maine and Vermont in their respective 1957 legislative sessions, but similar legislation was defeated by one vote on the final day of the 1957 New Hampshire Legislature. Present indications are that the proposed State-aid legislation will have the necessary support in the 1959 Session.

National Technical Task Committee on Industrial Wastes

The National Technical Task Committee on Industrial Wastes held its ninth annual meeting in Cincinnati, Ohio, December 10-12, 1958, attended by fifty-two scientists and engineers representing major industries in the United States. The Task Committee continued to direct its program to both the immediate and long-range industrial waste problems of the Nation.

The scope of the Committee's activities during the year is indicated by the subjects on which its Task Groups and Subcommittees reported at the annual meeting:

1. Research of special interest to industry, with priority recommendations from an opinion poll conducted during the year
2. Inventory of current and planned industrial waste studies
3. Information on noteworthy developments by industry in waste utilization, treatment, and disposal
4. Accelerated amortization as a means of encouraging progress in waste treatment
5. Plans for stimulating interest and encouraging training of competent personnel for meeting manpower needs in industrial waste control programs
6. Action on findings and recommendations from a study of municipal-industrial treatment problems
7. Publication of an article on a punched card system developed for recording industrial waste treatment and water quality improvement information in the literature
8. Program for improved communications through NTTCIW News Quarterly and other means, and development of a financing program to provide support for these activities
9. Suggestions on studies planned by the Sanitary Engineering Center
10. Development of a speakers bureau, with further encouragement and aids for regional and local industrial waste conferences, workshops, etc., for solution of problems in specific areas
11. Cooperation in basic data activities, including establishment of a subcommittee to advise and assist the Public Health Service
12. Completion and submission for publication of waste guides on the cotton, and cane sugar industries; along with a revised edition of the guide for the milk industry

Tannery Waste Report Released

The fifth publication of the New England Interstate Water Pollution Control Commission, in connection with its program of industrial waste surveys and research relating to pollution control of waters in the NEIWPC Compact area, has been released. This 41-page report is entitled "Tannery Wastes—Pollution Sources and Methods of Treatment." It contains a general description of the processes used in the tanning industry and the volume and pollution load contributed by each process. The analyses of over 75 samples of various process wastes together with the annual consumption and biochemical oxygen demand of the process chemicals used by tanners are tabulated in the report. One chapter is devoted to the procedure in conducting a tannery waste survey to determine the pollution loads, and another to the methods employed in treating tannery wastes.

This report is the result of a two-year study by the Industrial Waste Laboratory of Wesleyan University at a cattleskin tannery and a pigskin tannery in New England. The project was sponsored by NEIWPCC with a program grant under the Federal Water Pollution Control Act as the first step toward the development of practical and economic methods for the treatment of tannery wastes.

Copies of the report have been distributed to all the tanneries in New England and New York State, and to pollution control agencies, industrial waste consultants and technical libraries. Copies may be obtained from the Office of the Commission at 73 Tremont Street, Boston 8, Mass.

Ocean Disposal of Cannery Waste

National Canners Association personnel are studying the effect on marine conditions of the disposal at sea of a strong waste liquor which accumulates as drainage from a land-fill disposal of solid refuse from fruit and tomato canneries.

An estimated 1200 tons per day of wet cannery refuse is deposited in earthen cells near the shore of San Francisco Bay, with a daily drainage of 100,000 to 150,000 gal. In past years this liquid was discharged through a pipe line extending into the bay. Because of the pollution problem created, an improved method of disposal has been sought.

Through the cooperative efforts of the canners concerned, the Regional Water Pollution Control Board, and the California Fish and Game Commission, an experimental program of ocean disposal of the liquid wastes has been worked out. Preliminary results of studies to determine the conditions created in the wake of the barge discharging the waste have been encouraging according to Walter A. Mercer of the National Canners Association.

International Conference on Waste Disposal in the Marine Environment

The Sanitary Engineering Research Laboratory of the University of California, Berkeley, and the California State Water Pollution Control Board are planning jointly to sponsor an International Conference on "Waste Disposal in the Marine Environment" to be held on the Berkeley campus in June 1959.

The objectives of this Conference will be:

1. To bring about a critical review and appraisal of research in marine pollution problems in California and throughout the world,

2. To stimulate interest and research in marine problem areas related to waste disposal, and
3. To promote cross-fertilization between related scientists and engineers of many disciplines, agencies, and organizations.

Those interested in attending this Conference or obtaining more information should communicate directly with Professor Erman A. Pearson, Sanitary Engineering Research Laboratory, University of California, Berkeley 4, California.

REVIEW OF ACTION TAKEN UNDER THE ENFORCEMENT PROVISIONS OF THE FEDERAL WATER POLLUTION CONTROL ACT*

Hearings (P.L. 845)

Corney Drainage System (Arkansas-Louisiana)

Operators of 75 oil well leases in the Corney Drainage System were cited by the Public Health Service as discharging oil well brines to the waters of the Corney Drainage System so as to interfere with their use for stock-watering, irrigation, and other agricultural uses, sport and commercial fishing, recreation, municipal use, most industrial uses, and use by a valuable species of wild animals inhabiting the area.

Voluntary corrective action was obtained at 57 of the leases. Pollution from the remaining 18 leases was the subject of a Public hearing held by the Public Health Service at Homer, Louisiana, on January 16 and 17, 1957. On the basis of the findings, conclusions, and recommendations made by the Hearing Board, the Secretary of Health, Education, and Welfare sent notifications to the operators of these 18 oil well leases directing them to cease and desist from contributing to the pollution of interstate waters of the Corney Drainage System.

Inspections by Public Health Service personnel reveal that all operators of the 18 leases have ceased their polluting discharges. The Service is currently keeping this area under continuing surveillance in order to determine improvement of water quality resulting from the abatement of pollution.

Formal Conferences

Big Blue River (Nebraska-Kansas)

Municipal and industrial sources of pollution in the State of Nebraska—primarily in Beatrice, Nebraska—were discharging their wastes into the Big Blue River and interfering with its use as a source of public and industrial water supply; for irrigation, stock-watering, and fish habitat in the State of Kansas.

At a conference held by the Public Health Service on May 3, 1957 in Beatrice, Nebraska, it was agreed by the conferees that the offending communities in Nebraska should undertake the construction of remedial sewage

*Includes hearing held under P.L. 845, the Water Pollution Control Act of 1948.

treatment facilities. Beatrice, Nebraska has completed construction of a new sewage treatment plant which will accommodate the city's industrial and municipal wastes. Six of the other Nebraska communities which were cited as sources of pollution have applied for Federal grants to aid in financing new sewage treatment facilities. Two of these, Milford and Hastings, now have their plants in the process of construction.

Missouri River—St. Joseph, Missouri (Missouri-Kansas)

The city of St. Joseph, Missouri, together with its associated industries, was discharging large quantities of municipal, meat-packing, and chemical wastes to the Missouri River so as to degrade the quality of the water and interfere with its use as a source of public water supply, industrial water supply, commercial fishing, livestock watering, and recreational purposes in the State of Kansas.

A conference was held by the Public Health Service on June 11, 1957 in St. Joseph, Missouri. The conferees agreed that the community and its industries should construct sewage treatment facilities. The Missouri Division of Health agreed to report to the conferees at specified intervals on progress toward the achievement of the goals set at the conference.

Missouri River—Omaha, Nebraska (Nebraska-Iowa-Kansas-Missouri)

Municipal and industrial wastes from a number of communities in the Omaha, Nebraska area, primarily from the cities of Omaha, Nebraska and Council Bluffs, Iowa, were polluting the Missouri River so as to interfere with its use as a source of public water supply by St. Joseph, Missouri; for industrial water supply, recreational uses, stock-watering, commercial fishing, and fish and wildlife habitat in the States of Nebraska, Iowa, Kansas, and Missouri.

On June 14, 1957, the Public Health Service held a formal conference at Omaha, Nebraska, at which the conferees agreed upon a time schedule calling for remedial action by all the sources of pollution. Omaha, Nebraska, specifically, should initiate construction of a new sewage treatment plant to be under contract not later than January 1, 1960. The conferees also agreed that Omaha should provide for the improvement and expansion of an existing waste treatment plant and that Council Bluffs, Iowa, should install sewage treatment facilities. Progress reports from Nebraska and Iowa on the achievement of the objectives set at the conference are to be provided the conferees at six-month intervals until adequate waste treatment facilities are completed.

Potomac River—Washington, D. C. Metropolitan Area (District of Columbia-Virginia-Maryland)

Inadequately treated municipal wastes from Washington, D. C., Arlington County, Fairfax County, and Alexandria, Virginia, in addition to untreated storm water overflows from the combined storm and sanitary sewers in the District of Columbia, were degrading the Potomac River in the Washington, D. C. Metropolitan Area so as to interfere with the use of adjoining land areas as sites for residential, sight-seeing, and recreational purposes, as well as to render the waters of the river unsuitable as a game fish habitat. In addition, these wastes constituted a health hazard to commercial and recreational users of the river in the States of Virginia, Maryland and in the District of Columbia.

The first session of a Public Health Service conference was held on August 22, 1957. The conference was recessed to permit study and evaluation of a forthcoming report "Comprehensive Master Plan on Pollution Problems of the Potomac River", as well as to provide an opportunity to plan for the establishment of a time schedule for taking steps to abate the waste discharges from the several communities.

The second session of this conference was held on February 13, 1958. The conferees agreed that facilities should be added to the Washington, D. C. sewage treatment plant to provide secondary treatment by the end of 1958 with further plant improvements to be completed by 1965. In addition, Plan "C", a program of separation of storm sewers from sanitary sewers in the District of Columbia is to be completed by the end of 1966. Virginia communities are to keep pace with the District in the degree of treatment accorded its waste discharges.

Missouri River—Kansas Cities Metropolitan Area (Kansas-Missouri)

The three communities which make up the Kansas Cities Metropolitan Area—Kansas City, Kansas; Kansas City, Missouri; North Kansas City, Missouri; and local industrial establishments—were discharging municipal and industrial wastes to the Missouri River without treatment, degrading the quality of the river's water and interfering with its use as a source of public water supply in a number of communities in Missouri as far downstream as St. Louis County, Missouri. These wastes also created a health hazard to commercial and recreational users of the river; prevented its full use as a commercial fishery or for sport fishing; interfered with navigation and the actual and potential uses of the river for many industrial purposes; created severe sight and odor nuisances which have prevented the use of various river front sites as recreational areas and parks; and interfered with use of the river as a wild fowl habitat.

On December 3, 1957, the Public Health Service held a conference at Kansas City, Missouri. The conferees agreed that the offending communities and their associated industries should provide adequate treatment for their wastes, and that the first construction contracts for these remedial facilities should be let not later than January 1, 1960. The Kansas State Board of Health and the Missouri Division of Health will report at six-month intervals on progress toward the achievement of the goals set at the conference.

Mississippi River—St. Louis, Missouri Metropolitan Area (Illinois-Missouri)

Municipal and industrial wastes discharged to the Mississippi River from the St. Louis, Missouri Metropolitan Area, which includes communities in Missouri and Illinois, have caused deterioration of the quality of the river's water so as to interfere with its use as a source of public water supply, for recreation, scenic values, pleasure boating sport fishing, navigation, and commercial fishing in both Missouri and Illinois.

The Public Health Service held a conference on March 4, 1958, at St. Louis, Missouri. The conferees agreed on a time schedule for the construction of waste treatment facilities by the offending sources of pollution composed of cities, political subdivisions, institutions and industries on both the Missouri and Illinois sides of the Mississippi River.

Animas River (Colorado-New Mexico)

A uranium milling industry at Durango, Colorado, has been discharging wastes containing radioactive substances to the Animas River. In addition, the communities of Silverton and Durango, Colorado have been discharging municipal wastes, and mining operations in the Silverton Area have been discharging mining wastes into the stream.

A conference was held by the Public Health Service on April 29, 1958, at Santa Fe, New Mexico. The conferees agreed that further study of the stream would be necessary before the pollution problem could be defined. Hence, the Colorado Department of Health would prepare a report of non-radioactive waste for the other conferees by July 1, 1959, and the Public Health Service would undertake an investigation of the radioactive aspects of the pollution problem with the participation of New Mexico and Colorado. Upon its completion, no later than April 29, 1959, the second conference will be called with a view to evolving definitive conclusions and recommendations.

Missouri River—Sioux City, Iowa Area (South Dakota-Iowa-Nebraska)

The municipalities of Yankton, Vermillion, and Elk Point, South Dakota, Sioux City, Iowa, South Sioux City, Dakota City, Ponca and Blair, Nebraska and their associated industries were discharging their wastes into and causing pollution of the Missouri River. This pollution endangers the health and welfare of persons in Iowa and Nebraska.

A conference was held by the Public Health Service on July 24, 1958, at Sioux City, Iowa. The conferees agreed that pollution of the Missouri River, caused principally by municipal and industrial wastes from Sioux City, Iowa, interferes with its use as a source of the public water supply of Omaha, Nebraska; creates a health hazard to recreational and navigational users of the river; interferes with navigation, creates sight and odor nuisances, and creates conditions inimical to fish and aquatic life. In addition, the conferees agreed on a time schedule for the construction of waste treatment facilities at Sioux City, Iowa and at other sources of pollution.

Special counsel representing Sioux City, Iowa, who attended the conference as a participant, declared that the city would not comply with the recommendations of the conferees to construct waste treatment facilities on the grounds that Sioux City wastes were creating no pollution problems subject to abatement under the Act, and furthermore, that such construction was not economically feasible.

Lower Columbia River (Washington-Oregon)

Wastes discharged to the Columbia River by a variety of industries, predominantly pulp and paper mills, together with inadequately treated municipal sewage from Vancouver, Washington, Portland, Oregon and a number of other smaller communities in both States, are creating problems of pollution in the river. These are (1) bacterial contamination which may be a potential health hazard to commercial and recreational users of the river, and which interfere with the water's use as a source of public water supply and (2) the growth of slimes which create conditions deleterious to commercial and sport fishing.

The first session of a conference was held by the Public Health Service on September 10 and 11, 1958, at Portland, Oregon. The conferees agreed that the States of Washington and Oregon would undertake a joint program to

control pollution of the river and that a second session of the conference would be convened not later than September, 1959 to review the progress of the joint undertaking, and to make definitive recommendations for further action as necessary.

Bear River (Idaho-Utah-Wyoming)

Municipal and industrial wastes, consisting primarily of beet sugar wastes, were being discharged from sources both in Idaho and Utah and were finding their way to the Bear River. The resulting pollution of the river interfered with aquatic life and wildlife; made it unsuitable as a potential source of public water supply; interfered with the use of the Cache River Basin as a recreational area; and created a potential health hazard when the river's water was used for irrigation and stock watering.

The first session of a conference was held by the Public Health Service on October 8, 1958, at Logan, Utah. The conferees agreed that the State of Utah, with the assistance and advice of the Public Health Service, is to gather information which will permit Utah to classify the waters of the Bear River in accordance with Utah State law. The second session of the conference will be held following that classification of the river, for the purpose of establishing a time schedule for remedial measures to be taken by sources of pollution in Idaho and Utah.

* * *

AIR POLLUTION

National Conference on Air Pollution Reports

A limited number of copies of "Highlights of the National Conference on Air Pollution"—a 48 page booklet—are available upon request to Mr. Thomas F. Williams, Information Officer, Air Pollution Program, Public Health Service, Washington 2, D. C.

The Conference Proceedings, which will include the complete text of all formal presentations and also the principal informal comments made in discussion sessions and during the "Open Forum" will be available about March 1. These can also be requested from the Public Health Service in Washington, D. C.

National Council for Stream Improvement Studies Atmospheric Pollution

The initiation of a project at the College of Engineering of New York University to prepare an Air Pollution Manual dealing primarily with particulate matter, brings to three the number of research grants made by the National Council for Stream Improvement to study the atmospheric disposal problems of the pulp and paper industry. This project, under the direction of Prof. W. T. Ingram, seeks to assemble and compose, under one cover, essential information which will be helpful to individual mills burning solid fuels in complying with the air pollution regulations now being promulgated. Some of the subject headings of the manual are: Legislative Aspects of Air Pollution, Methods of Compliance with Code Provisions and Measurement of Atmospheric Pollutants.

Good progress is reported for the National Council's other two projects in this field. Prof. Donald T. Adams, head of research at Washington State College, has developed a gas chromatographic technique for the separation and identification of eight sulfur-containing compounds which have potential application to the study of gaseous pulp mill effluents at the sources and in the atmospheric vicinity of the sources. Work is continuing on developing colorimetric techniques for analyzing sulfur-bearing gases which might be applicable for use in the Division of Industrial Research's continuous, automatic recorder. The Council's kraft industry study project, under Professor E. R. Hendrickson's direction at the University of Florida, has been working in the field with Southern kraft mills on simplified methods of stack gas analysis. This involves determination of hydrogen sulfide, sulfur dioxide, and total sulfur, as well as particulate matter. Development and laboratory testing with regard to the collection efficiency and interferences with the method is now underway.

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